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THE WATER WORKS OF DENVER, COLORADO.

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READ SEPTEMBER 20TH, 1893.

WITH DISCUSSION.

In order to present intelligently the interesting features of recent construction in the new water works of the city of Denver, with which the writer has for three years been connected in the capacity of consulting engineer, it may be of interest to give first a brief historical account of the development of the old works and thus explain the conditions which led to the construction of a new system.

The first works, built in 1871, consisted of a pair of Holly pumps, with a capacity of 2 500 000 galls. daily, drawing water from a large well sunk in the gravel beds of the Platte River, at the foot of Fifteenth Street, then the main thoroughfare of the city. The town had then but 6 000 inhabitants, but was growing rapidly, and two years later an additional pump with 2 000 000 galls. capacity was added, which sufficed for public needs till 1879, when a higher source of supply was sought above the sewage contamination of the spreading city. The new works were located in West Denver, two miles above the original location, where it was possible to utilize the water-power of

the river for pumping purposes. A canal, some two miles long, so wide and capacious as to be dignified by the name of Lake Archer, was constructed from the river to the new works, where a vertical fall of 30 ft. was available. A battery of turbine wheels and a set of four Holly horizontal pumps, having a combined capacity of 5 000 000 galls. daily, constituted the new plant, to which were added two more Holly pumps with 5 000 000 galls. additional daily capacity, in 1881. The water-power being insufficient at times for the increased duty, a 350-H. P. compound engine, with corresponding boilers, was also added at the same time. Both sets of pumps derived their supply directly from Lake Archer, and while the volume was adequate, the quality was not altogether satisfactory. The lake, in time, accumulated a great mass of silt and decaying vegetable matter, which began to contaminate the water, and in 1884 the water company decided to seek a purer supply still higher up the stream, by developing the underflow in the gravel beds adjacent to the river, on the opposite side, and 3½ miles above the works. There were then no factories or settlements of material size above the intake, the military post had not then been located, and the city had not grown to within threatening distance of the source of supply; hence, the site selected was a good one for the time being. The development of the new supply was planned and executed by Mr. C. P. Allen, chief engineer of the company, and was the originating motive for the evolution of the wooden stave pipe patented by him, which has since come into extensive use. The grade from the gravel beds to the works was so slight that a very large pipe was needed to carry the volume of water required, and the maximum pressure upon it was so low that there seemed to be an especially good field for the employment of some material very much cheaper than iron. It was finally decided, after considerable experimentation, to lay a wooden stave pipe, 48 ins. in diameter, banded with round rods, the top half to be of California redwood and the lower half of Colorado pine. The cribs or galleries which gather the water and feed the pipe were made of rough lumber, 4 ft. in width, and 1 to 4 ft. high, with open joints, the tops of which are some 6 ft. below the level of the low-water surface of the river. Some 1 400 ft. of these cribs were placed, and resulted in a minimum yield of 12 000 000 galls. daily, which increased to 18 000 000 galls. during flood stage. The water, filtering through gravel, was clear and pure, and when the works were

completed there was a vast improvement over pre-existing conditions. The pipe terminated in a 12 000 000-gall. reservoir at the works, whence it is pumped into the mains. This still constitutes the principal source of supply of the Denver Water Company, although the yield of the cribs has been very much reduced by the waste from a large paper mill that pours its sewage into the river a few hundred feet above them, forming a film of paper pulp over the bed of the stream that interferes with the free percolation of water through the gravel. To remedy this defect, in a measure, deeper cribs were sunk next to the channel of the stream in the beginning of 1892, and a centrifugal pumping plant of large capacity was erected to lift the supply some 10 to 15 ft. into the wooden conduit. This arrangement necessitates a double pumping of the supply; and it is only a temporary remedy, as the strong draft from the new cribs has been gradually choking the gravel to a greater degree; and whereas, at the outset, the centrifugal was only needed during low-water stage in the river, it has of late been run without cessation.

The success attained in developing the underflow of the Platte encouraged the company to attempt a similar development on Cherry Creek, a tributary stream that joins the Platte River in the heart of the city. This stream heads in the Great Divide, south of the city, and has a minimum slope of 30 ft. per mile. It has a large water-shed area (380 sq. miles) of plains or prairie lands, and a wide, deep bed of sand, and is subject to torrential freshets, but has a very small normal flow, as it has no mountain drainage and is not fed by perennial snows. Its slope rendered it practicable to bring out its underflowing waters in a comparatively short conduit and deliver them to the city mains at a sufficient elevation to avoid the necessity of pumping. In the winter of 1886-87 a wooden stave pipe, 37 ins. in diameter, five miles long, was laid to tap a system of semi-circular cribs sunk in the bed of the creek at a maximum depth of 25 ft. below the surface, or about 22 ft. below the normal water surface. These cribs were tight on top and sides, but open at bottom, this construction being preferable in quicksand. They cross the channel from bed-rock on one side, within 700 ft. of the outcrop, to bed-rock on the other, and hence drain all the underflow to the depth to which they are laid. Borings showed the extreme width of this subsurface channel to be about 2 000 ft.; maximum depth, 41 ft.; area of sand cross-section about 62 000 sq. ft., of which about 40%

was above the level of the cribs. The lower end of the pipe terminated at a covered reservoir, excavated in earth, with a capacity of 7 000 000 galls., located on Cemetery Hill, in the southeastern suburbs of the city, the level of high-water surface being 5 417 ft. above sea level. At the time of its construction this was the limiting height of city pressure, but in 1891 the pressure was increased throughout the city some 20 lbs. (giving 100 to 110 lbs. pressure throughout the business quarter), and since then the Cherry Creek supply has also to be pumped. When the 37-in. pipe was completed, the flow reached a maximum of 9 000 000 galls. daily. After four months this began to decrease, and in one year had diminished to 2 000 000 galls. daily, which is about the normal yield since that time. This yield is at the rate of about 80 galls. per day per square foot of cross-sectional area drained by the cribs, or about 2 500 galls. per square foot of bottom area of the galleries or cribs.

The quality of this water has always been better than that of the Platte, but both have been affected by the largely increased area of irrigated land, the seepage from which, returning to the stream, carries with it a strong solution of alkaline salts that render the water hard and objectionable to the taste. These streams of seepage water are so considerable in volume that, according to recent gaugings by the State Engineer of Colorado, the water returning to the Platte in this way above Denver amounted to 96.38 cu. ft. per second in November, 1891, at the close of one irrigating season, and in March, 1892, at the beginning of the next, it was 84.57 cu. ft. per second, ranging from 47 to 56% of the total flow of the river at the time of the gaugings. The water is frequently darkly stained, a brownish color, especially during the irrigating season, and in that state is brackish and unpleasant to the taste; in fact, quite unfit to drink until it mingles with the larger volume of the river. Irrigation is indispensable to the welfare of the country, but it is frequently accompanied by this disadvantage, that it does not improve the quality of potable water where it is practiced extensively.

In 1890, The American Water Works Company, successors to The Denver Water Company, began active operations with the object of meeting the competition of The Citizens' Water Company, organized the preceding year, by a large extension of its mains in the city, an increase of its pumping capacity by the erection of two high-duty Gaskill pumps of 12 000 000 galls. daily capacity, and the development

of an additional water supply. The scene of operations for the latter work was on Cherry Creek, some distance above the head of the 37-in. pipe, where, during the summer of 1891, some $900 000 was expended in preliminary preparations for drawing a vast water supply, sufficient for all present and future needs of the city, from the sand beds of the creek. Unfortunately, financial embarrassments prevented the completion of these works before any water was developed, as their operation would have been of great scientific interest. A large timber caisson, some 800 ft. long, 16 ft. wide and 40 ft. deep, was to have been sunk across a portion of the channel, which was to have served as a receiving well, from which water was to be pumped to a large reservoir constructed on a hill half a mile distant, 150 ft. above the level of the stream. This reservoir has a capacity of some 95 000 000 galls., and was built with unusual disregard to expense, the embankments being supported inside by massive double walls of masonry, standing vertically 20 ft. high, the space between them being filled with thin cement grout. Some sixteen heavy pressure 48-in. valves are inserted for various functions in the walls. Rows of handsome brick and cut-stone dwellings for the use of employees were erected. The general scale of operations was evidently based on the expectation that this would be the chief, if not the sole, source of supply for the future, and that upwards of 50 000 000 galls. was to be made immediately available, which was the amount frequently mentioned in press interviews as the quantity to be at once obtained. Much doubt has been expressed by engineers of good repute, especially by those who have had occasion to experiment on the velocity of water percolating through sand, as to the possibility of the existence of any very considerable volume of underflow in this stream, even approaching in amount the quantity above stated, and on account of the extraordinary expenditure made prior to a conclusive demonstration of the volume of probable flow, it has seemed to the writer to be of sufficient interest to call special attention to the case.

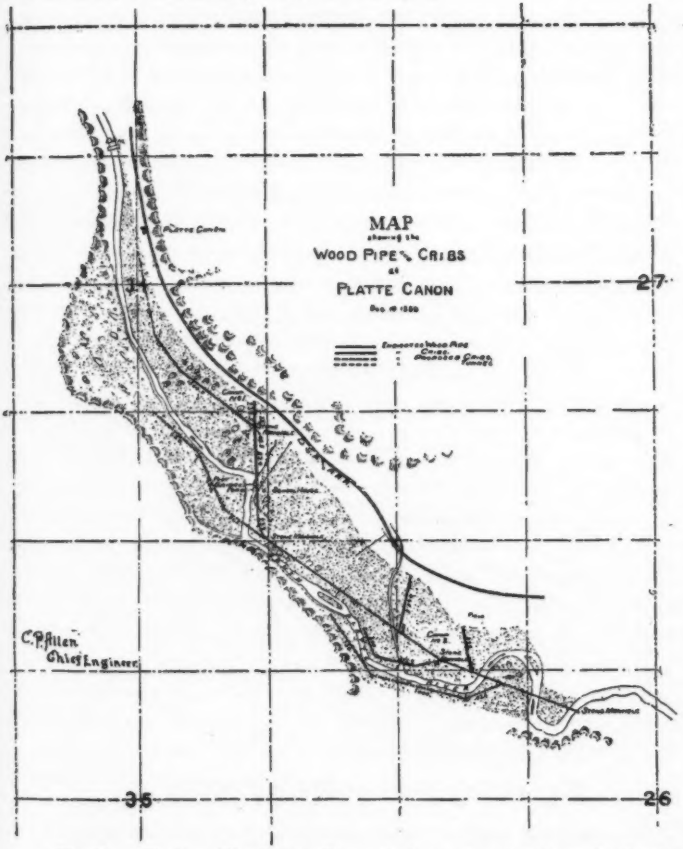
But little positive data seem to have been recorded as to the rate of underflow or percolation through sands on various slopes, but all notes on the subject that the writer has been able to collect seem to agree fairly well within limits of from $\frac{1}{2}$ mile to 2 miles per annum, the velocity being apparently influenced less by the slope than by the degree of fineness of the sand. Applying the maximum of these veloci-

ties to the case in point, the cross-sectional area of sand required to pass even 40 000 000 galls. daily, assuming the voids to be 30% of the mass, would be 610 000 sq. ft., or, say, 3 miles wide by nearly 40 ft. average depth, which is 10 times the area of cross-section determined by borings at the cribs a short distance below, and 19 times the area presented by the side of the proposed caisson, heretofore referred to. The writer has been so deeply impressed with the wide-spread fallacies entertained by many intelligent people on the subject of fabulously great and "inexhaustible" volumes of water presumed to be flowing beneath the surface of sandy beds of western streams that he wishes to provoke discussion on the subject, and elicit an expression of the experience and opinions of engineers, either in opposition to, or corroboration of, his views on this question. In his opinion, a visible, tangible water supply, stored up in a capacious reservoir, is always preferable to an invisible, immeasurable, conjectural supply lurking hidden in a bed of sand that is chary about giving it up, especially when the one is as easily and cheaply secured as the other, as was the case at Denver.

The Works of The Citizens' Water Company.

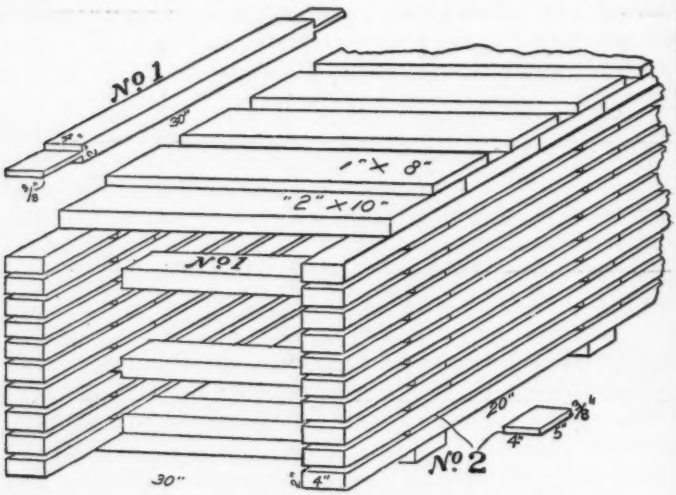
The fact that over 400 artesian wells were bored by private parties, at an aggregate cost of over $500 000, within the city limits of Denver, from the time of the discovery of this source of supply in 1883 until its limited volume was fully demonstrated some years later, is a significant commentary on the quality of the general water supply at that time, and is evidence of a desire to obtain purer water. The Citizens' Water Company was organized March 29th, 1889, by a few wealthy citizens of Denver, with the avowed purpose of meeting this public demand. Its organizers had been minor stockholders in the old water company, and had endeavored to impress the majority holders with the necessity of adopting a gravity system of supply from a pure source in the mountains. Failing in this, and being more deeply interested in the continued growth of the city than the majority seemed to be, they finally sold their stock and organized a rival company. Thoroughly conversant with the details of the business and the general problems of the situation, they entered into the enterprise with confidence in the ultimate outcome. Enlisting the services of Mr. C. P. Allen, Chief Engineer, and Richard Holme, General Manager, of the old company,

they lost no time in beginning operations, and within 15 months from the date of their organization had completed a 30-in. gravity pipe line from Platte Cañon to the city, 20¼ miles long, had laid some 70 miles of mains in the city and secured a large share of patronage, fighting their way inch by inch through injunctions, harassments and personal encounters, whose history would fill volumes.



The New Works.—The Platte River, before emerging from the Rocky Mountains and entering upon the plains, passes through many miles of rock-bound cañons, where its waters are so thoroughly aerated in

tumbling over a succession of cascades that any possible impurities that might have been imparted to them by the sparse settlements on its upper sources become neutralized and eliminated. At the point of exit from the mountains the stream has deposited great beds of gravel, covering several hundred acres, which are naturally saturated with water that sinks in the bed of the cañon above. To obtain this water for the preliminary supply, the company purchased a large area of these gravel beds and set to work to drain them. Beginning at a point whose elevation was sufficient to admit of the free flow of water therefrom to the highest points in the city limits, trenches were dug on light grades following up the general course of the valley, with several lateral branches. In these trenches were constructed rectangular wooden cribs or galleries, 30 ins. square, formed of 2 x 4-in. rough lumber on the sides, with ¼-in. spaces between them, the tops solidly planked over and the trenches refilled. The total length of main trunk cribs is 3 584



Plan of Crib-Work of Citizens' Water Co.

ft., with 328 ft. of laterals. Their depth below surface is 14 to 22 ft., and their total cost, including five masonry manholes with valves, was $47 807 90, an average of $6 89 per foot. The total excavation required was 45 212 cu. yds. The result of this work was a measured flow of

3 260 000 galls. of pure, clear water, which remained constant for many months, until the galleries were flooded to increase their yield up to the capacity of the pipe—8 400 000 galls. daily. Gaugings at the different manholes indicated that the yield of the cribs at 22 ft. depth was nearly three times that at 13 ft. Four large ponds were formed over the galleries, and water turned into them from the river. They cover about 20 acres and are cleaned out twice a year, the deposited silt being scraped out and the surface of the gravel loosened by plowing. From them the water percolates freely to the drainage galleries, where it emerges perfectly filtered and clarified.

The 30-*in. Conduit.*—From the cribs of the Ashland Avenue distributing reservoirs, located some four miles west of the center of the city, a 30-in. conduit was constructed, the total length of which is 107 012 ft., or 20.26 miles. Of this distance, the greater portion, 87 580 ft., or 16.4 miles, is made of wooden stave pipe. Cast-iron pipe was required to be used for a distance of 17 145 ft., passing through the streets of the municipality of Highlands. A tunnel on the line, 2 288 ft. in length, 36 ins. in diameter, lined with concrete, completes the entire length. The wooden pipe is banded to withstand a maximum pressure due to 185 ft. static head below the hydraulic grade, which extreme occurs near its junction with the iron pipe. At the crossing of the valley of Bear Creek, it lies 150 ft. below the grade line for more than 1 000 ft., and at numerous points it withstands from 50 to 100 ft. head of pressure. The total number of bands used was 271 900, which, divided by the length, gives an average spacing of 3.86 ins., which is sufficient for an average head of 70 ft. throughout. This, however, was much more than was required for the grade line from the present source of supply, the extra banding having been added with a view of increased pressure to be given to the line by a proposed branch line from a large reservoir that was to be constructed at a high level near the head of the pipe.

The cost of this 16.4 miles of wood pipe was very moderate, as shown by the following detail:

1 869 000 ft., B. M., Texas pine, surfaced, at $27.50	$51 399 28
271 900 ¼-in. steel bands and shoes	54 299 55
Erection of pipe, at 5.1 cents per band (contract)	13 866 03
Total	$119 564 86

This is an average of $1.36½ per foot. Trenching and backfilling cost 48.3 cents per foot, having been rendered unusually expensive by several miles of cutting from 10 to 30 ft. deep, and one river crossing, where the channel was turned and the pipe laid several feet below the bed of the stream.

The condensed profile of the entire conduit, Plate IX, will show at a glance the general topographical features of the line and the pressures borne by the wood pipe in the numerous dips below the grade line. The general course of the conduit is north, and for the greater portion of its length it is on the west side of the Platte River.

Although its actual terminus is at the reservoirs mentioned, connection is made with the city distributing system at various points from two to five miles above the reservoirs. Being thus freely attached to the city mains, with open connection with the reservoirs, only the surplus over and above the consumption at any time passes into the reservoirs, which occurs chiefly at night. Whenever consumption temporarily exceeds the volume of normal flow in the conduit, the deficit is drawn from the reservoirs, and thus for the lower two miles or more the water flows alternately into and out of the reservoirs through the conduit. This varying condition renders it impossible to fix an absolute hydraulic grade, but that which is shown upon the profile is assumed to terminate at the point where the first large lateral taps the conduit. The capacity of the conduit to the reservoirs, with city connections closed, is 8 400 000 galls. daily at the maximum, or 6 300 000 galls., with reservoirs at full stage. The measurement has been made both volumetrically, *i. e.*, by observing the depth added to the reservoirs in a given time, and by velocity measure with a current meter at a manhole in the tunnel. The results indicate that in applying Kutter's formula to wood pipe, as low a coefficient n as .0096 can be used, although we have generally assumed $n = .010$ in all calculations of capacity. The wood pipe, having a smooth interior which becomes smoother with use, has a decidedly greater carrying capacity than cast-iron pipe of equal diameter, the difference amounting to about 16% as far as deductions are warranted from observations covering a limited range of diameters.

The Second Conduit.—Within a year after the completion of the first conduit, it became evident that another of still greater capacity was immediately required to supply the growing demand for "mountain

Condensed Profile
of
30" Conduit.

PLATE IX.
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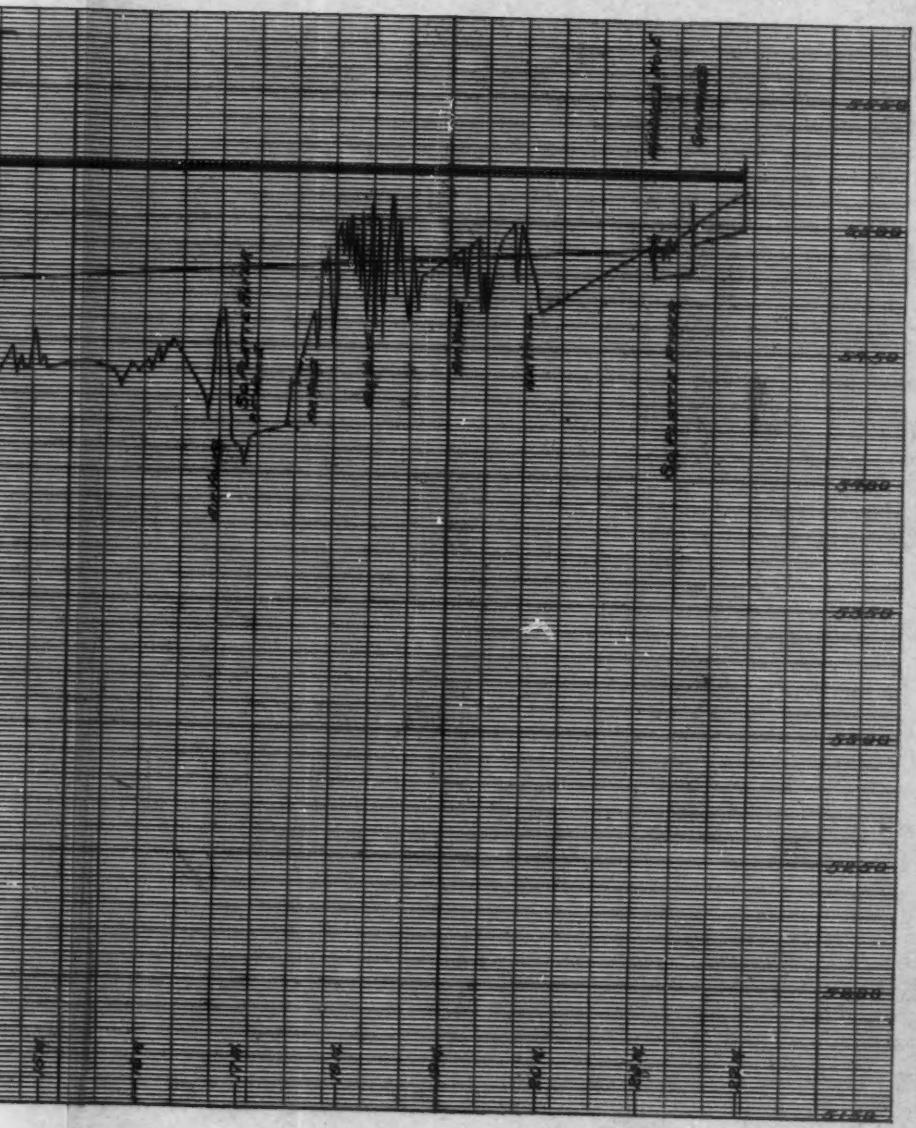


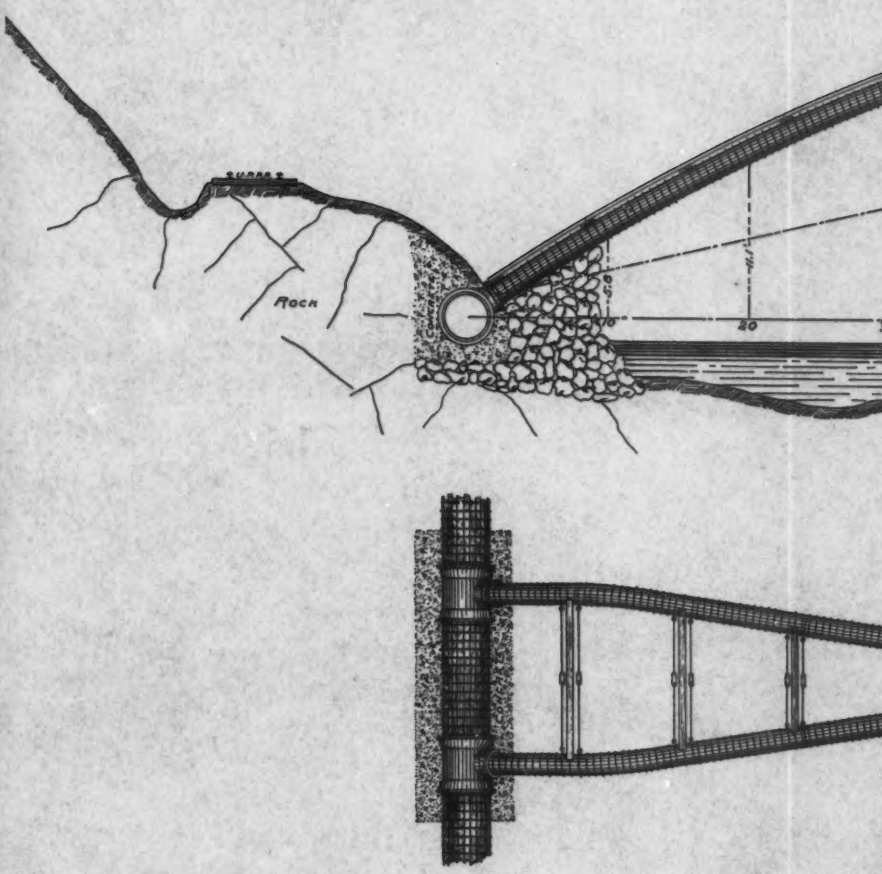
water." The general plan proposed from the beginning of the enterprise had been to construct a conduit of 60 000 000 galls. daily capacity from the Forks of the Platte, some 15 miles above the mouth of the cañon, to a large storage and settling reservoir located within the "hog-backs" at the foot of the mountains, some two miles west and 500 ft. higher than the head of the first conduit, and thence, by a short pipe connection with the latter, increase its capacity by the addition of 100 ft. or more head, to the extent of probable requirements. It was in pursuance of this plan that the pipe was so heavily banded, as heretofore mentioned. The location of this great conduit along the cliffs of the cañon was made and work begun on the dam, but as the dam and conduit were estimated to cost about $1 000 000, and required three years or more to complete them, and no addition to the supply could be expected from any quarter until both were completed, it became apparent that the delay was too great a sacrifice of time. This led to a change of plans, and it was finally resolved to build a second conduit directly from the river, and filter the water, starting high enough up in the cañon to give a grade that would deliver 16 000 000 galls. daily through a 34-in. pipe. It was to start on the left or west bank of the stream and follow parallel with the first conduit, but on higher ground, for about half its length, then to cross the valley of the Platte through the town of Littleton and enter Denver on the highest ground to the east, on the opposite side of the city from Conduit No. 1. It was considered highly advantageous in a gravity system to place the city between two large feeders to its mains. This general plan was carried out and the conduit completed in April, 1893, although a material change was made at the head as the result of the purchase of a large natural basin, locally known as Marston Lake, to be used as a storage reservoir. This is located about nine miles in a direct line from the geographical center of Denver, and may in the near future be within the city limits. It is 1½ miles northwest of the nearest point on the line of the second conduit, where the latter turns eastward to cross the Platte. A connection was made from the lake to the second conduit by means of a 44-in. wood pipe, which was extended from the connecting point east for 2 000 ft. as a part of the main conduit, to where the latter crosses Conduit No. 1, with which it was also connected at the crossing. Thus the lake can be drawn upon to supply both lines through the one 44-in. pipe, giving to them a

combined capacity of 30 000 000 galls. daily. The elevation of the reservoir at high water line is 5 545 ft. above sea level, or 350 ft. above city base, which was too high to permit its being fed from the river through the 34-in. conduit; and a third and larger conduit is to be built to supply the lake. This will necessarily start 1½ miles higher up the cañon than the originally located head of Conduit No. 2, and it was decided to make but one diverting dam for both conduits, and make but one conduit large enough for both lines through the principal part of the cañon work, or to a point known as Tunnel 1.

The location for the filters for the second conduit was selected on property owned by the company some four miles below Tunnel 1. Between the filters and the city the hydraulic grade was established for the desired capacity, but between Tunnel 1 and the filters the difference in level was sufficient to give a capacity of 18 000 000 galls. daily to a 30-in. pipe, thus giving an ample surplus over the capacity of the 34-in. line, for use in washing the filters. Accordingly, the diameter was reduced to 30 ins. above the filter plant.

Tunnel 1 cuts through a sharp granite ridge, which is also pierced by the High Line Canal Company, whose tunnel is 25 ft. lower and about 100 ft. distant at the north end. Pending the completion of Tunnel 1, and the dam and aqueduct above, a temporary arrangement was made with the High Line Canal Company to use their tunnel and take water at a waste gate at its mouth into the 30-in. pipe. A short flume was built from the waste gate, with sand gates and regulators, and connected with the 30-in. main on the opposite side of the river by means of two 13-in. wooden pipes built in the form of an arch spanning the stream. The entire span from the center of flume to the center of the 30-in. pipe is 104 ft. These pipes are spread apart 12 ft. at the ends and are brought together at the center for a distance of 20 ft. where they are tied together with iron bands. This construction, which was made as a cheap and temporary crossing, is novel, but is stanch and effective, and illustrates what may be done with wood pipe. Plate X shows the plan of the structure, from which a slight deviation was made owing to the fact that the pipes refused to bend exactly to the lines of the drawing, but assumed curves of their own. Plate XI is a side view looking down the river.

The arch was built on light false work and was finished in less than a week from the time the materials were assembled. When the false


Rock
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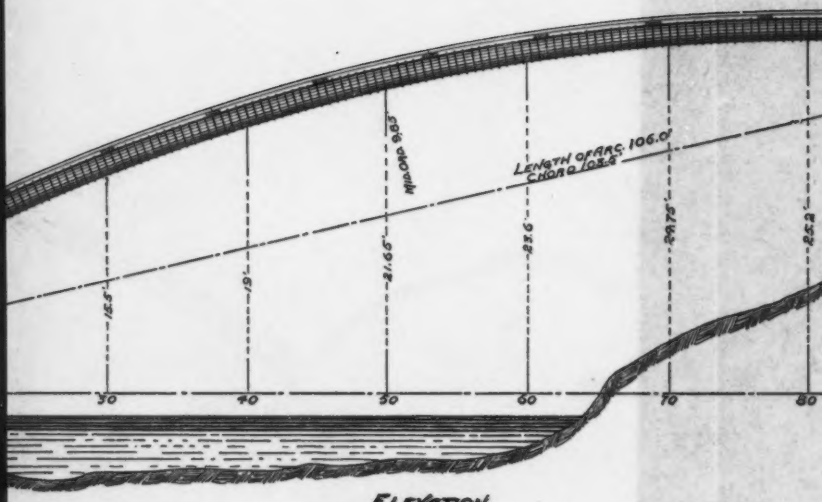

ELEVATION

PLAN

PLATE X.
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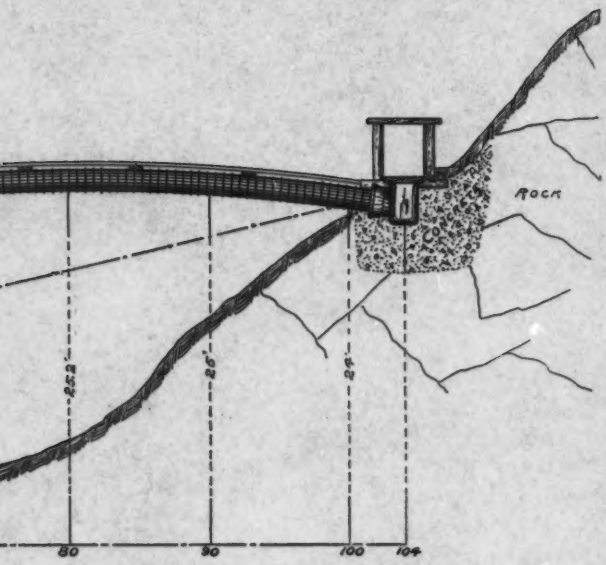
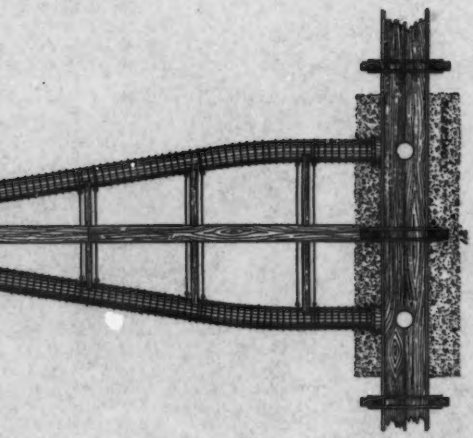
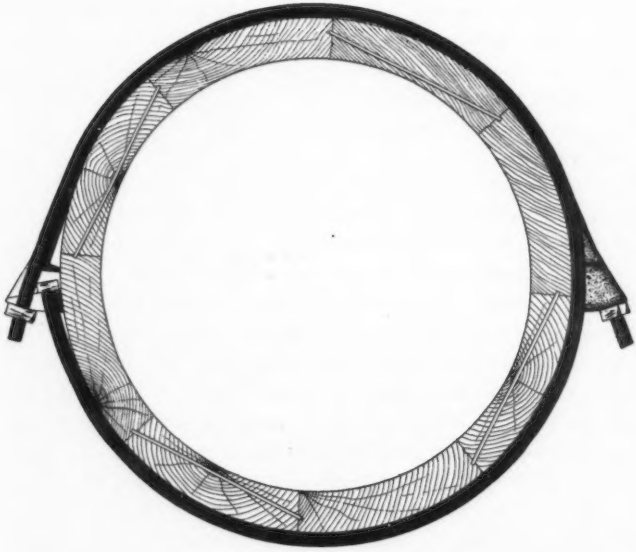




work was removed and the pipes were filled with water the maximum deflection was but $\frac{7}{8}$ in. with eight men standing on it. It is constantly used as a footbridge, and has but little vibration, even in a gale of wind. The bands around the pipe are $\frac{3}{8}$ in. diameter, spaced evenly $2\frac{1}{2}$ ins. apart. The lumber used is Texas pine, $1\frac{1}{4}$ ins. thick, in 12-ft. lengths. In this case the end joints were made every 6 ft., each alternate stave extending to the next joint. The special castings at each end were set in a heavy bed of concrete, filling the space between them and the solid rock in place, the whole interwoven with small steel rods. The arch has no tension member and one end is 24.5 ft. higher than the other, which is another unusual feature, although one which cannot be said to add to its value.



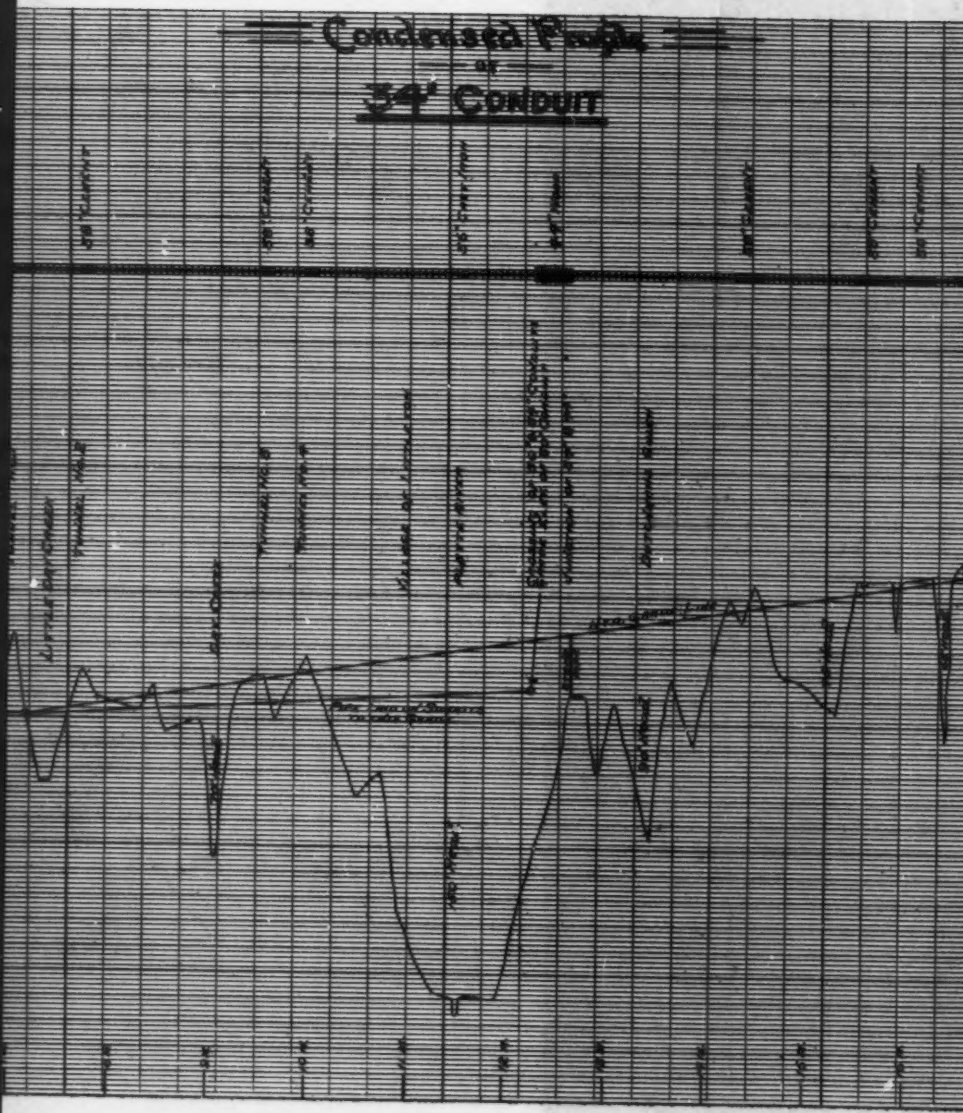
A somewhat similar arch occurs at another point on the line where the 34-in. pipe crosses a canal and is unsupported for about 60 ft. In this case, the versed sine is about 3 ft. As it was made without a drawn plan no definite data can be given of it, but it is simply mentioned as a further illustration of the ease with which short spans can be

made, it having been constructed precisely like all the rest of the pipe.

A condensed profile of the second conduit is presented in Plate XII. At all the high points where the hydraulic grade is below the ground surface, a pipe of Portland cement concrete was constructed, either in open trench or tunnel. The remainder of the line is of wood pipe, except the crossings of Platte River and Cherry Creek, where 36-in. cast-iron pipe was used. California redwood was employed for the greater portion of the wood pipe, and a part was built of long-leaf yellow pine from Texas.

The fact that the entire water supply of a city of 150 000 inhabitants is conveyed in wooden mains of this type is so radical a departure from all precedents that it is deserving of more than passing notice. It is manifestly and unreservedly successful, and has achieved an enormous saving in cost; and if successful and economical here, there is no reason why it should not be equally so in other localities, nor why its merits should not meet with general recognition, freed from preconceived notions of its alleged temporary nature and lack of durability.

One of the strong features of the wooden stave pipe is that it is built continuously in the trench where it is to lie, and not in sections made in the shop; hence, it involves no difficulties in handling, transportation, loading and unloading, lowering into the trench, etc., as with other classes of pipe, which become serious in the ratio of their diameter. A defect in a stave or a band or a shoe does not involve the rejection of a section. Being made continuous, of staves of irregular lengths, it has no defined end joints, although the sections are generally joined within a space of 2 or 3 ft., over which space it is customary to distribute three extra bands wherever the spacing is less than 150 bands per 100 ft. The butt joints are "staggered," so that no two are nearer each other than 6 ins., and from that to 3 ft. The staves are dressed in the planer to conform to the true curve of the pipe, both inside and out, and hence are of uniform thickness and must be evenly saturated when the pipe is filled and in use. To insure saturation from the inside, they are made as thin as possible consistent with strength, rigidity and ability to hold up under a load of cover. The edges are dressed to exact radial lines, with a minute tongue or calking bead $\frac{1}{16}$ in. high on one edge of each stave. When the pipe


Condensed Profile
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34" Conduit

PLATE XII.
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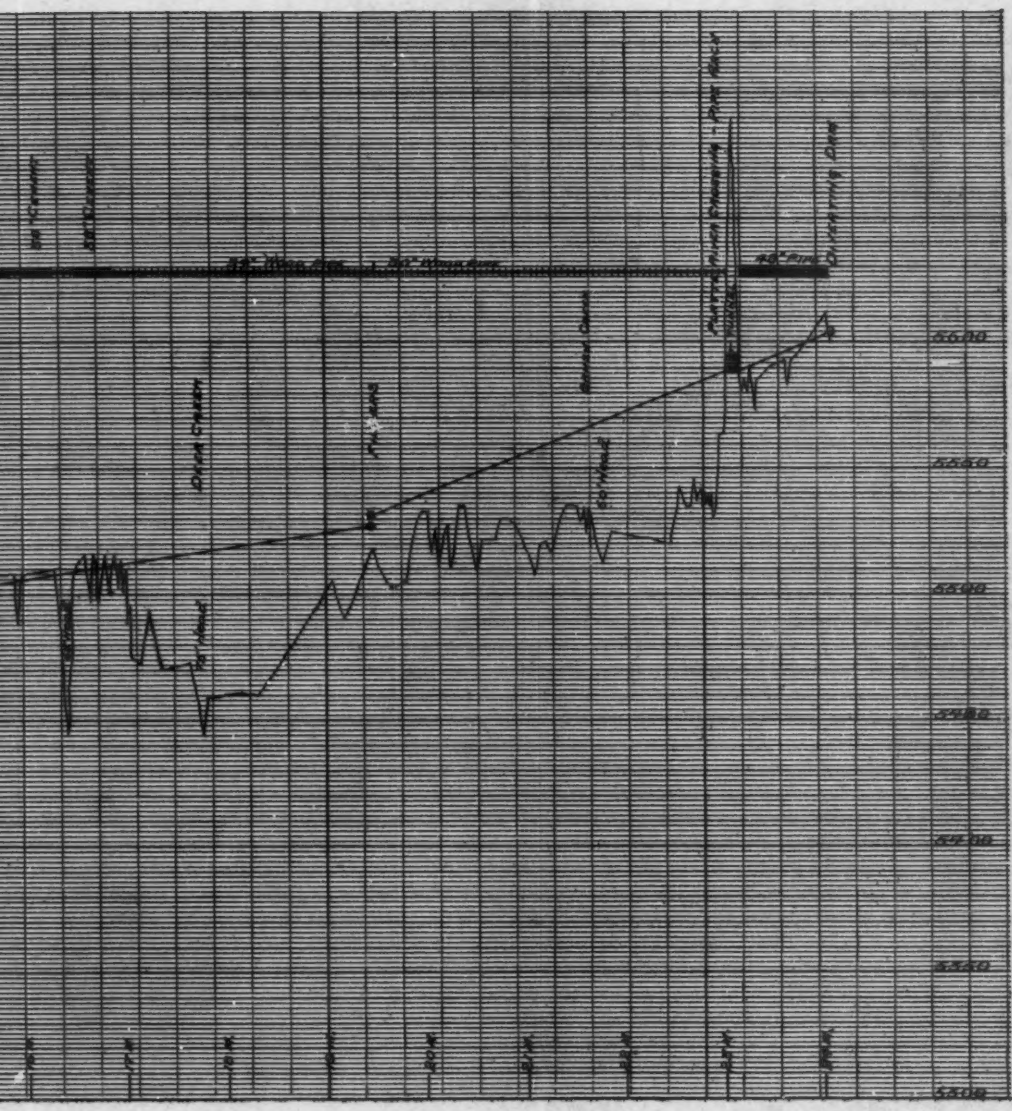
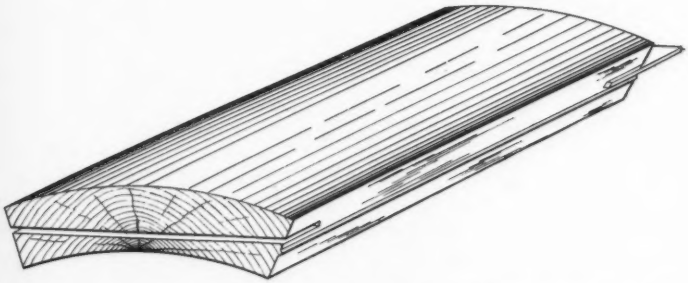
SCHUYLER ON WATER WORKS OF DENVER.



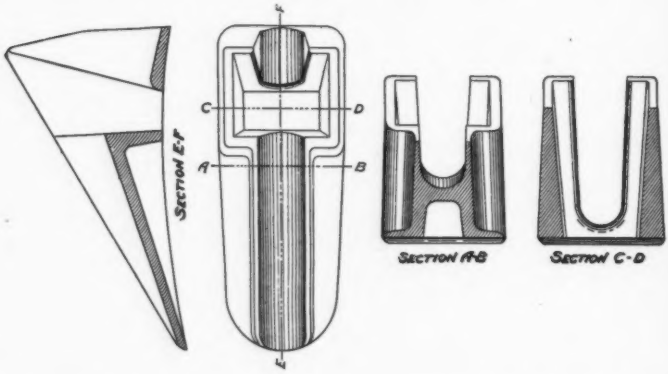
is tightly drawn together, this is embedded in the adjoining stave, and serves to take up any slight irregularity in the planing of the edges. Leaks seldom or never occur along the longitudinal seams.



Metallic tongues are inserted in the ends of the abutting staves ¾ in. deep by means of a saw kerf. These tongues are cut ¼ in. longer than the width of the stave, and thus project ⅛ in. on each side, the projection being embedded into the wood of the adjoining stave when the bands are tightened. This serves to cut off leakage around the ends, which would otherwise occur whenever the corners of the staves were slightly jammed or shrunken. The end slots to receive the tongues must be cut with precision, both as to depth and position, to avoid irregularity of surface of the pipe, both inside and out.

The bands used are made of mild steel, having a tensile strength of 58 000 to 65 000 lbs. per square inch, and an elastic limit of 40 000 to 50 000, with an elongation of 24 to 27% in 8 ins. They are generally made 4 to 5 ins. longer than the exterior periphery of the pipe, it being desirable to have considerable spare length to facilitate the rapidity of the work of erection, as too much time would be lost in "trying to make both ends meet," if the bands were barely the right length. They are made with a head on one end, and nut and thread on the other, the threads being cut 5 ins. It has been found advantageous to have the threaded ends upset so as to make the bands of equal strength throughout, as recently improved processes render it feasible to upset small rods for a length of 5 ins. The ends of the bands are brought together with a cast or malleable iron shoe of peculiar construction having a slot near the bottom to receive the head end of the band, the other end dropping in over the head between the two jaws, against

which the nut and washer set, producing a straight and direct tension on the band. Malleable iron shoes are preferable to cast iron, as the latter may be easily broken by a careless blow when under tension; and as the malleable shoes can be made very much lighter, their cost is little, if any, greater. Malleable shoes were exclusively used on the second conduit. Their weight is but 1½ lbs. each. One pattern was used interchangeably on the 30, 34 and 44-in. pipe.



Beginning at the bottom, the staves are set up inside a U-shaped mould, usually a piece of 1¼-in. gas pipe. When the half diameter is reached, an inside form, completing the circle, also of gas pipe, with the ends lapping and spread apart so as to stand alone, is placed inside the half pipe, and the requisite number of staves added to complete the full pipe. The foreman is furnished with a profile on which the number of bands for each station is marked, and he marks on the top of the pipe the exact position of each band, spacing them according to the profile. Then the bands, which have previously been bent around circular bending tables of proper diameter and coated with paint or dipped in a hot bath of asphalt, are slipped on over the ends of the staves, distributed to position, the shoes attached, and the nuts partially screwed up. While the bands are being adjusted, lined up perpendicular to the pipe and partially tightened, the pipe is rounded out and "coopered," to bring all the staves out full, and the staves are carefully driven home to meet the abutting staves. The driving of the staves is repeatedly gone over as the bands are tight-

PLATE XIII.
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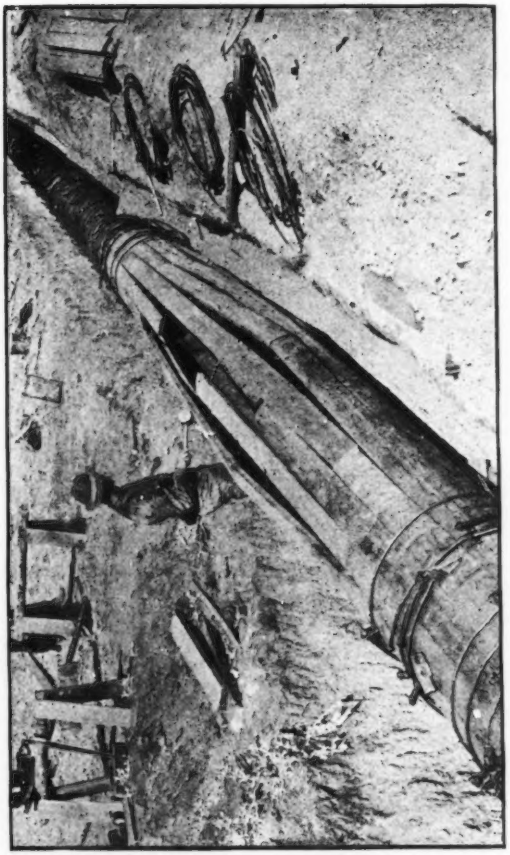


ened. The final "cinching" of the pipe before it is covered with earth is usually deferred till the day after it is laid, when every nut is gone over and brought up to extreme tension. Brace wrenches are used with levers not over 10 ins. long, as a precaution against over cinching. It is desirable to draw the bands tight enough to imbed them slightly into the wood, but where they are but 2 or 3 ins. apart great care must be taken to avoid crushing the wood or collapsing the pipe. This frequently occurs, and has been the cause of the most serious leaks when the pressure came to be applied. If the staves are quarter sawed and the grain runs obliquely across, the excessive tightening is apt to cause a split and a lapping of the two parts, which might escape detection until the pressure was on.

A pipe-laying gang consists of from 8 to 16 men, according to the number of bands per unit of length. One-half the gang is usually employed in back-cinching. The speed of laying is dependent upon the number of bands to be placed, as well as upon the diameter. On the 34-in. pipe the gangs usually placed 700 to 1 000 bands per day, laying from 150 to 300 ft. of pipe. On the 44-in. pipe the rate was about 500 bands per day. The cost of erection is from 5 to 10 cents per band, on pipe from 30 to 48 ins. diameter.

It is customary to distribute the gangs 1 000 to 2 000 ft. apart, and where they meet the coupling is made with ease, the last staves being cut $\frac{1}{8}$ in. long and sprung into position; thus the buckle joint (see Plate XIII) is the tightest of all. This facility of construction renders it possible to lay a line very rapidly when the trench is ready and the materials are distributed.

To make good pipe it is essential that only the best of lumber be used—clear, straight-grained, without knots, or checks, or season cracks, or flaws of any kind. It should also be thoroughly dry and seasoned before milling. Then, after it is milled, the sooner it is built into the pipe the better. If milled when only partially seasoned it is apt to shrink unequally in width, and more at the ends than at the center. This end shrinkage causes troublesome joint leaks. The shrinkage in California redwood, that had seasoned 60 to 90 days before milling, was frequently as much as 3 ins. in the 20 staves that formed a 34-in. pipe. This had to be met by the insertion of a narrow stave of varying width to fill the full diameter, as the bands are of an exact length and cannot be made to accommodate the lumber.

The formula used for the distribution of the bands was:

$$N = \frac{1\,200\ dp}{2s}$$

where N = number of bands per 100 ft.,

d = diameter of pipe in inches,

p = pressure in pounds per square inch,

s = safe working strain in pounds, for bands when threaded for use, determined by regular tests at the mills where they were made.

In making up the tables used for the distribution of the bands, values of s were taken as follows:

$\frac{3}{8}$-in. bands,	plain	s	=	1 000
$\frac{3}{8}$ " "	upset	s	=	1 200
$\frac{1}{2}$ " "	plain	s	=	2 000
$\frac{1}{2}$ " "	upset	s	=	2 500
$\frac{5}{8}$ " "	plain	s	=	3 000
$\frac{5}{8}$ " "	upset	s	=	3 500

These values of s give a factor of safety of about 5 in each case. or about one-fourth of the elastic limit.

Changes in the rate of spacing are made with every 5 ft. change in piezometric head, with the exception that for low heads the limit of distance from band to band is fixed at 17 ins.

All the high-pressure pipe in use that has been examined by the writer was found in a condition of complete saturation, with moisture oozing slightly over its entire surface. For perfect preservation from decay this seems to be the ideal condition. Numerous samples have been taken from pipe in use from three to nine years in this vicinity, and in every instance it has been found perfectly sound. Hence it is fair to assume that the wood of the pipe is likely to be as long lived as cast iron, so long as it is kept in this condition by constant use. The bands are the only perishable portion of the pipe, and these, when properly coated, should be good in ordinary soils for 15 to 20 years.

Bands that have been in place nine years do not yet manifest any serious corrosion. Moreover, they may be easily replaced without interrupting the use of the pipe, and it is a simple matter to replace a defective stave here and there.

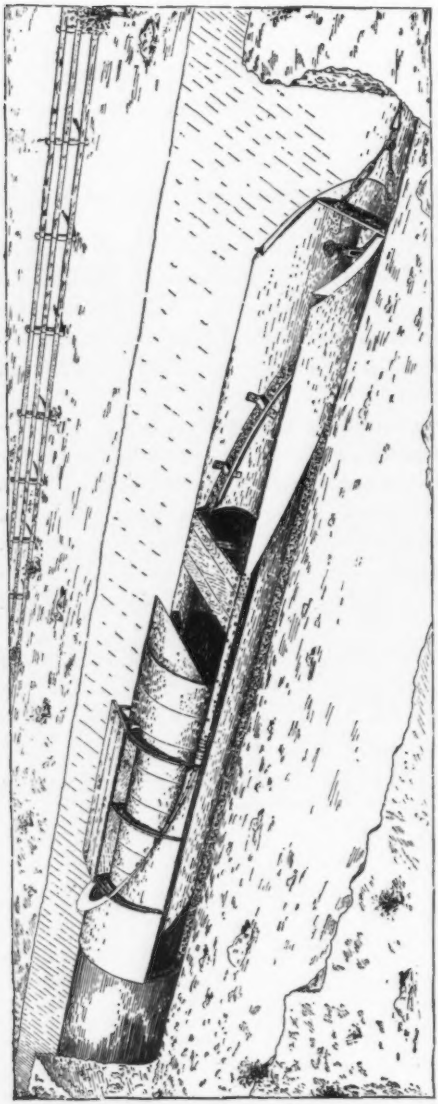
One advantage possessed by the pipe is quite evident, especially when open at the lower end; it can never burst, as cast iron frequently does, without apparent cause or warning, and sometimes with disastrous damage to property. It may spring a leak, but it gives ample warning of any serious weakness. Leaks are sometimes allowed to run for months, and they never increase materially and give no anxiety. A few pine wedges are usually all they require, and sometimes a toothpick will serve.

If the pipe can be allowed to fill very slowly and soak for weeks under little or no pressure before the full head is turned on, there will be very few leaks developed subsequently, if the work of erection has been properly done and good materials used. Extreme care and inspection of every stave, seam and joint in construction is well repaid when the pipe finally comes into use.

Outlets for laterals of over 6 ins. diameter are usually made by means of cast-iron tees, through which the wood is built, the opening of proper size being sawed out afterwards. The bell ends are made about 5 ins. deep, with a space of ¾ in. for oakum calking. For small connections a cast-iron saddle is strapped around the pipe, with lead or rubber gasket underneath. For ordinary house services, from ½ to 2 ins. diameter, a hole is bored into the wood and the service cock screwed in.

At a moderate estimate the saving effected by The Citizens' Water Company by the use of wooden pipe for their main conduits has been no less than $1 100 000 over the cost of cast-iron pipes of equal capacity. The interest on this amount at 6% would renew the mains every five or six years, or duplicate them as often as that if necessary. With such a showing as this, the correctness of which cannot be challenged, there can be no doubt of the soundness and wisdom of the judgment which governed the selection of wood as the material for the main conduits of the Company. If this be admitted, why may not the experience of this case be repeated anywhere, with equally favorable results?

Cement Concrete Pipe.—As the line of the second conduit follows generally the country roads, little choice could be made to secure uniformity of depth of cutting. Where the grade line was deeper than 14 ft. below, the high points were tunneled and lined with cement concrete 3 to 4 ins. thick. Where the depth was from 6 to 14 ft., an open trench was excavated and concrete pipe formed in the bottom. A large



portion of this work was done with a pipe mould invented by E. L. Ransome, of San Francisco. The machine consists of a wrought-iron shell of the size of the inner diameter of the pipe, to which was attached a "leader" and "saddle" of larger diameter, the space between which left a ring or core, that formed the pipe when filled with concrete. The mould was drawn along the trench slowly by a cable and horse-whim, and, as it moved, the concrete was shoveled and rammed into the core space on a long incline that gave room for tamping it throughout its circumference. The top half of the pipe was supported by sheet-iron plates, 2 ft. wide, that were placed one after another on the forward end of the mould, and, being clamped firmly together at the top and sides, remained in position while the mould slid out from under them. They were supported,

PLATE XIV.
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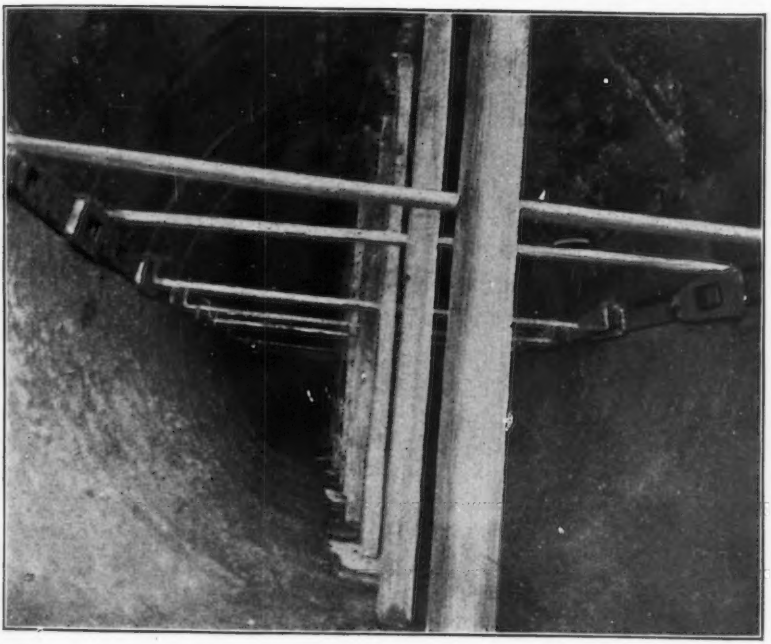

after leaving the mould, by sticks placed under the center of each, and held to an uniform width by small clamping rods (see Plate XIV). The plates were allowed to remain in from 24 to 48 hours, until the concrete had set sufficiently to be self-supporting.

A gang of 30 men operated the machine and performed the various functions of mixing, wheeling, shoveling and tamping the concrete, extracting the plates in the rear, bringing them forward, cleaning and greasing them, and putting them in position. When everything was in good working order and the concrete could be mixed and delivered without delay, the machine was capable of making about 600 ft. of pipe per day, and on short runs would go at the rate of 900 ft. per day; but counting all stoppages, the rate did not exceed 300 ft. per day. The inner diameter was 38 ins., and the thickness of shell $2\frac{1}{2}$ to 3 ins. The bottom was moulded flat for 18 ins. The concrete was mixed in a mill, also invented by Mr. Ransome, which automatically measured the cement, sand and gravel in the desired proportions, and delivered the concrete, thoroughly mixed, in a continuous stream at the rear, whence it was shoveled into wheelbarrows. It was mounted on a 4-wheeled truck, with a small engine to operate it, and moved as often as necessary to keep up with the work. The superiority of the quality of concrete mixed by machine over that mixed by hand was apparent at a glance, when the two were compared in finished work, although no experimental tests were made to determine the relative strength of each. The proportions used were $3\frac{1}{2}$ of river sand and gravel to 1 of cement.

The total length of concrete pipe laid on the line of the second conduit was 14 470 ft. in nine separate sections of 600 to 2 200 ft., of which five were tunnels with a total length of 2 730 ft. The amount constructed with the machine was 8 900 ft., including one tunnel 700 ft. long. In the machine-made pipe, the weight of the machine and the gang of men riding on it aids greatly in compacting the concrete. The smooth surface of the inner mould gives a neat finish to the bottom half inside, while the saddle dragging over the top trowels down the outer surface on top. The pipe was washed with pure cement applied with brushes after the removal of the plates.

The cost of the machine-made pipe was $1 35 to $1 50 per foot, with cement at $3 75 per barrel; gravel, $1 25 per cubic yard, and labor, $1 75 to $2 00 per day. This was after the men were broken to the routine of their respective duties, and everything working smoothly.

The hand-made pipe was moulded on wooden sectional forms, 3 ft. long, the bottom in one piece reaching to within 3 ins. of the center on each side, while the top form, hinged in the center so as to fold inward, completed the circle. The thickness was nowhere less than 3 ins., and generally 3½ to 4 ins. In the tunnels, the entire space between the mould and the solid walls was filled with concrete, giving a thickness of 4 to 6 ins. all around. The contract-price for excavation of tunnels was $1 25 per linear foot. Shafts were sunk at intervals of 150 ft. The lining followed the excavation as quickly as possible, and before disintegration and crumbling of the earth took place.

One section of the concrete pipe was laid over a succession of short fills and cuts, the maximum depth of fills being 13 ft. They were near the crest of a ridge, and none of the little gulches have water-sheds over ¼ acre in extent, so that no culverts through the banks were needed. The fills were made of coarse gravel, sand and clay, and packed firmly. To support the pipe from possible settlement, posts or piers of concrete were put down through the embankment to the solid ground beneath by boring holes with an 8-in. post auger and filling them with concrete, in which were placed twisted steel rods ¼ in. square. These piers were put in rows 2 ft. apart, and over them was laid a concrete floor 4 ins. thick, 2 ft. wide, also containing twisted rods laid in longitudinally near the bottom, to take the tension, with cross rods every 4 ft. The pipe was built up on the floor. The embankment was carried up on each side of the pipe and high enough to cover it 2 ft. in depth.

The sinking of the cast-iron pipe under the beds of the Platte River and Cherry Creek were interesting features of the construction of the second conduit. The former was 156 ft. long, the latter 350 ft. In the Platte River crossing, the stream was forced to scour out the trench under the pipe as it was lowered in one section, on screws between rows of piles (see Plate XV). Sixteen long screws suspended the pipe and held it precisely level, and, with a man at each screw, it was lowered with exact uniformity. The bed of the river consisted of coarse gravel, and there were frequent hard spots or reefs that required digging, to assist the action of the current. The shore ends had to be dug down by hand entirely, but the greater portion of the work was done by the stream. When the pipe was down, so that the water began to run over the top, a dam was formed of boards set on an incline, resting

PLATE XV.
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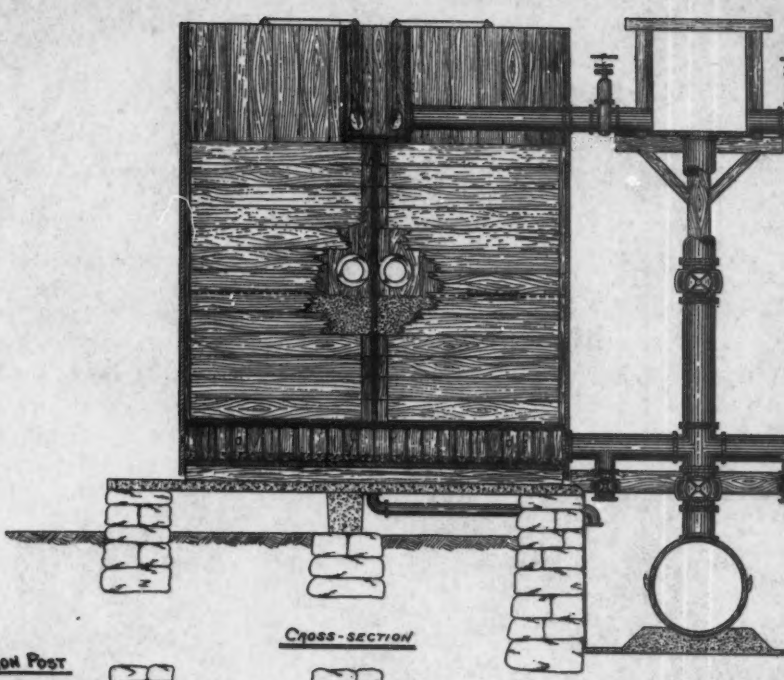
loosely against the pipe and the stringers on the piles (see Plate XVI). In this way the pipe was lowered to a depth of 2 ft. below the lowest part of the channel.

Another method was employed at the Cherry Creek crossing, where the stream bed is entirely of sand, with but little water flowing. A row of piles, 20 ft. long, was driven flush with the surface on the lower side of the pipe for it to rest against, and, when the joints were run and calked, it was lowered without suspending simply by digging the sand from under it, a little at a time, care being taken to excavate uniformly. To keep the trench free from water, a drain ditch was scraped out of the sand for a distance of about 1 000 ft. down stream. When the pipe had reached a depth of 4½ to 5 ft. beneath the surface, the joints were recalked and the trench allowed to refill, which it did at the first little freshet.

The two conduits cross each other at an angle of 61°, and here a somewhat novel connection was made, by means of which water from each of the mains can be turned into the other, or both be used independently. There are four valves in the cross, one in each of the main lines, and one in each of the by-passes. The entire cross, with its valves, was fitted together on the surface and the joints made with lead and calked, and the whole lowered in one piece with screws. Its weight was about 30 tons, and it rests in a bed of concrete that envelops it. Immediately below the cross Venturi meters are to be placed, to give a measure of the total volume of flow into the city.

The Filter Plants.—When the plan for the second conduit was adopted, and the mechanical filtration of its water supply was decided upon as a resultant necessity for at least a portion of the year, an investigation of the cost of the mechanical filters on the market led to a research for some cheaper way of accomplishing the same results on a large scale. The plans finally adopted, which are the invention of Mr. C. P. Allen, chief engineer of the company, bid fair to be, not only thoroughly successful, but in many respects an improvement upon the well-known types of filters in use, particularly in the matter of rewashing and cleansing the filtering material, while the cost of the plant per unit of capacity is but one-fifth to one-fourth the cost of efficient filters on the market. Plate XVII illustrates the general details of construction of the filters, which consist of a series of wooden tanks, 12 ft. in diameter, 14 ft. high, ranged in rows on either side of

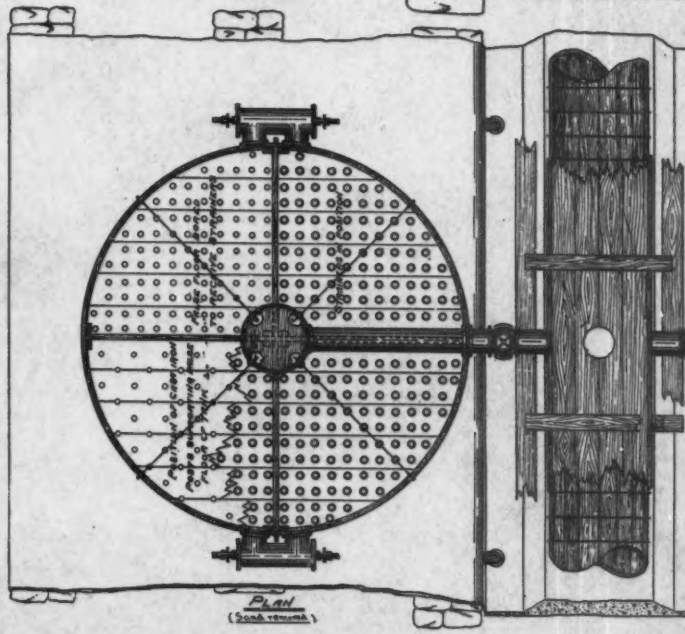
the main pipe. These tanks have a false floor of wood, 1 ft. above the bottom, supported and firmly attached to the bottom by cast-iron posts 1 ft. apart, with bolts running through both floor and bottom. In this false floor strainers of finely perforated brass plate are placed in holes bored through the floor 6 ins. apart, there being 500 strainers 2 ins. in diameter in each tank. The tank above the floor is divided into four compartments by wooden partitions reaching to within 4 ft. of the top. Clean quartz sand to the depth of 3 ft. is placed in each of the compartments. This is dried, screened and crushed, only that which passes a No. 20 screen and is retained on a No. 30 screen being used. The particles are therefore between $\frac{1}{400}$ and $\frac{1}{600}$ in. in diameter. Each compartment is provided with a valve for washing out the sedimentary deposit of filtration, this valve being placed about 18 ins. above the top of the sand. A valve in the main pipe, when closed, forces the water to rise into a flume passing the whole length of the plant at the level of the top of the tanks. Outlets with regulating valves connect the flume with each of the compartments of each tank. The filtration is downward, the clear water being collected between the floors of the tanks, whence it is conducted again into the main pipe below the main shut-off valve and continues on its way to the city. The washing of the filter beds is accomplished by a reversal of the current, and either filtered or unfiltered water may be used for washing. To use the filtered water for the cleansing process, it is necessary to throttle off a part of the clean water flowing from the bottom chamber to the main by partially closing the valve that controls it; then, by opening the waste valve to any one compartment, the water in that compartment is drawn down several feet lower than in the other compartments, and an upward current under pressure takes place through the sand from the chamber below, which is common to all the compartments. When the waste water runs clear, the waste valve is closed and that of another compartment opened, until all have been successively washed. The capacity of each tank is expected to be about 500 000 galls. in 24 hours at the minimum. This will require a velocity of about 5 ins. per minute through the sand, which is much greater than would be desirable or efficient in ordinary open-pond filter beds, but with the aid of a coagulent to ensnare and precipitate the particles of sediment, and by frequent washing—once or more daily—the rate given is not believed to be excessive.



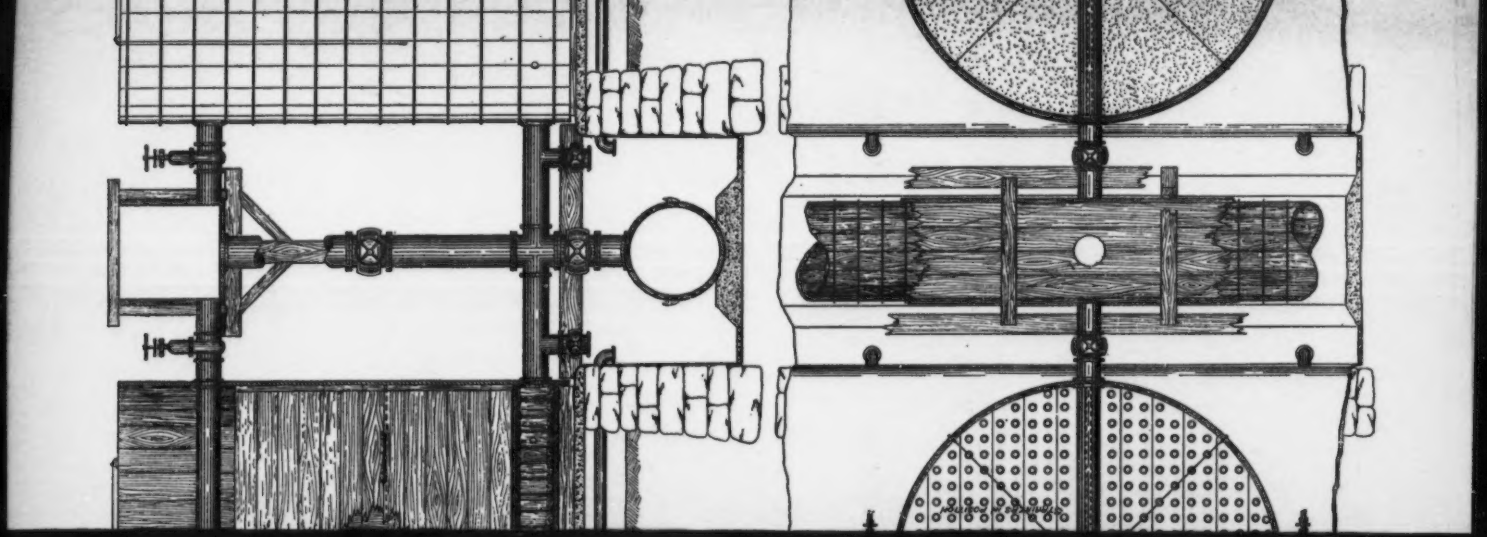
CROSS-SECTION

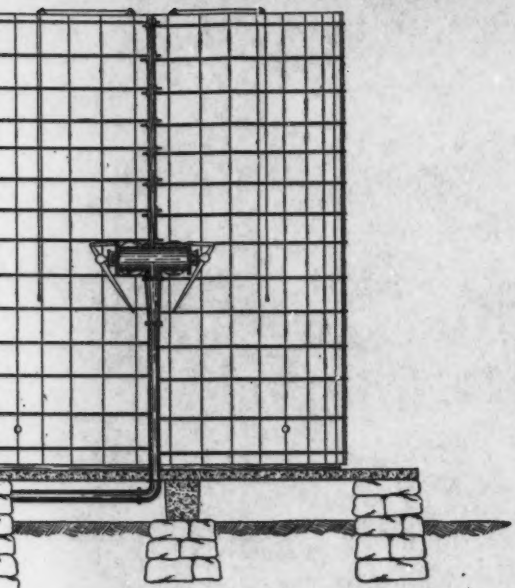
CAST-IRON POST

SECTION on A-B

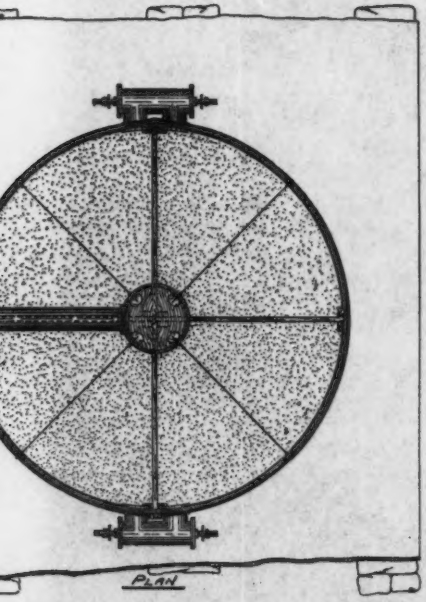


PLAN

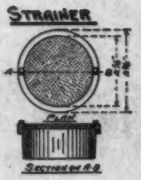




ELEVATION



PLAN



The plant has not been in operation a sufficient length of time to determine fully the duty of each tank on water of different grades of impurity, nor the cost of filtering the water, but it is estimated not to exceed $5 per 1 000 000 galls. at any time.

The foundations of the tanks consist of parallel lines of masonry walls, over which a 4-in. concrete floor is laid, with a grillage of ¾-in. square steel rods, 2 ins. apart in one direction, 3 ft. apart in the other, embedded in the concrete ¾ in. from the bottom.

At the main outlet to Marston Lake reservoir a similar filter plant has been erected, consisting of a battery of 10 tanks. The water of the lake, being comparatively free from sediment, will require but little effort or expense to render it clear, pure and sparkling.

A settling flume is in contemplation at the upper end of Tunnel 1, on the second conduit, to precipitate all sediment that will not be transported with a velocity of about 0.2 ft. per second. This will clarify the water to a considerable degree and increase the duty of the filters, as the mountain water settles quickly on being brought to rest, and seldom displays the milky cloudiness that is more troublesome to get rid of.

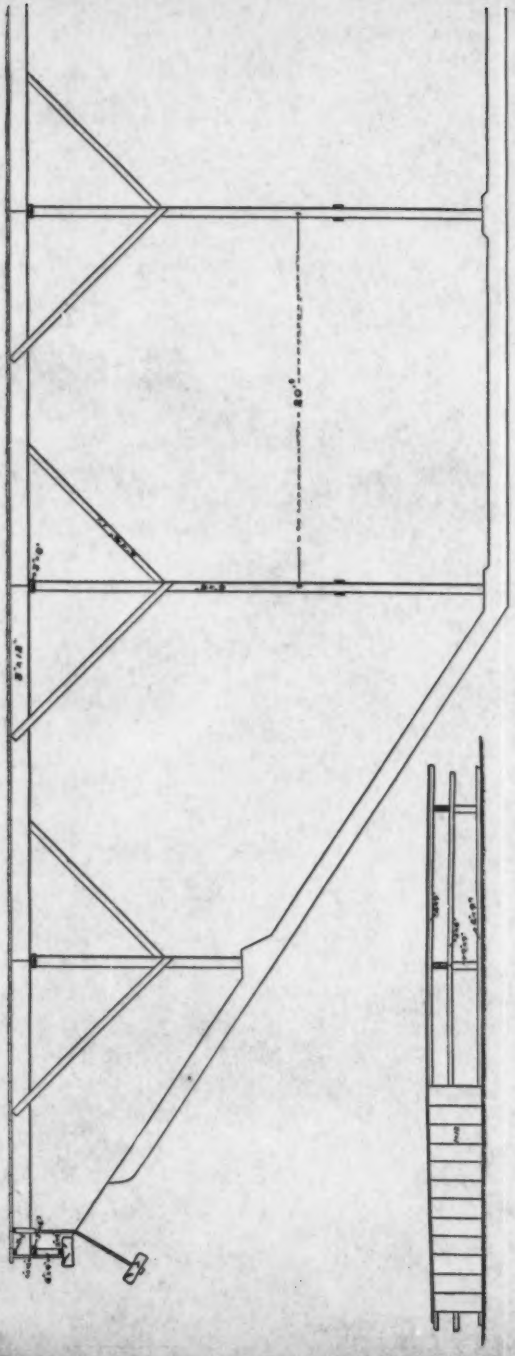
Reservoirs.—The first conduit ends in two distributing reservoirs on Ashland Avenue, some four miles west from the center of the city to which reference has already been made. A description of their asphaltum lining was given in Vol. XXVII, page 629, of the *Transactions* of the American Society of Civil Engineers. The reservoirs were covered with flat roofs of rough pine boards, supported by wooden posts 6 x 6 ins., placed in rows 20 ft. apart, and spaced 10 ft. apart in the rows (see Plate XVIII). The cover was not made tight, but was laid with spaces of 1 in. between the boards, which saved 10% in the lumber, without impairing the efficiency of the roof as a preventive of the growth of algæ, which was its sole object. It is perhaps needless to say that this purpose has been effectually accomplished. Panels at convenient intervals were arranged in the roof, to be lifted out when desired, affording openings for shoveling in snow whenever its depth on the roof became objectionable, as well as to give light, ventilation and easy access to all points in the reservoir when it is being cleaned. To facilitate a circulation of the water in the reservoir and maintain a constant movement in the mass of water, the pipe which serves as both inlet and outlet was extended, in the case of the east reservoir, around

two sides in the form of a square flume or box, tapering from 3 x 3 ft. to 1 x 1 ft. with 6-in. openings at intervals of about 50 ft. This was firmly fastened to the bottom, to prevent floating. With this device no portion of the water becomes stagnant, but is always kept in slow motion, whether the current be in or out.

On the opposite side of the city, near the terminus of the second conduit, two iron tanks have been erected on the top of an embankment 20 ft. high, the tops of which are level with the high water-line in the Ashland Avenue reservoirs. They are each 50 ft. in diameter, 45 ft. high, and have a combined capacity of 1 320 000 galls. They serve a useful purpose as equalizers to maintain supply and pressure to that part of the city, and to relieve the second conduit main pipe. They rest on a concrete base 2½ ft. thick, tied together with twisted steel rods.

Marston Lake Reservoir.—The main storage reservoir of the system is a natural basin near the foot of the mountains, known as Marston Lake. This basin, without any artificial embankments, has a capacity of 2 272 000 000 galls., covering 383.5 acres, and by the erection of dikes across two low gaps in the surrounding rim of the basin its capacity can be enormously increased; but a limit was fixed at 5 637 500 000 galls. as the ultimate capacity desired, which brought the high water-line 20 ft. above the lowest points on the gaps referred to. One of the dikes will be 3 300 ft. in length, the other, 2 100 ft., at a height of 5 ft. above the water-line. The material available is suitable for these embankments and can all be taken from the interior of the basin. The reservoir, when full, will cover an area of 647 acres and have a maximum depth of 67 ft.

The main outlet to the reservoir, the one connecting with the two conduits already built by the 44-in. pipe heretofore mentioned, is at the southeast corner of the lake, and consists of a tunnel 1 600 ft. long, driven to tap the lake at its lowest bottom. It is 38 ins. in diameter and is lined with concrete 5 to 6 ins. in thickness. The greater portion of its length is in yellow shale and soft sandstone. Flanges 12 to 16 ins. deep, 12 ins. wide, were cut entirely around the periphery of the tunnel at intervals of 50 ft. and filled with concrete during the process of lining, to interrupt percolation between the lining and the natural walls of the tunnel, although, considering the very firm nature of the material and the thoroughness with which the concrete was rammed into



80'

PLATE XVIII.
TRANS. AM. SOC. CIV. ENGRS.
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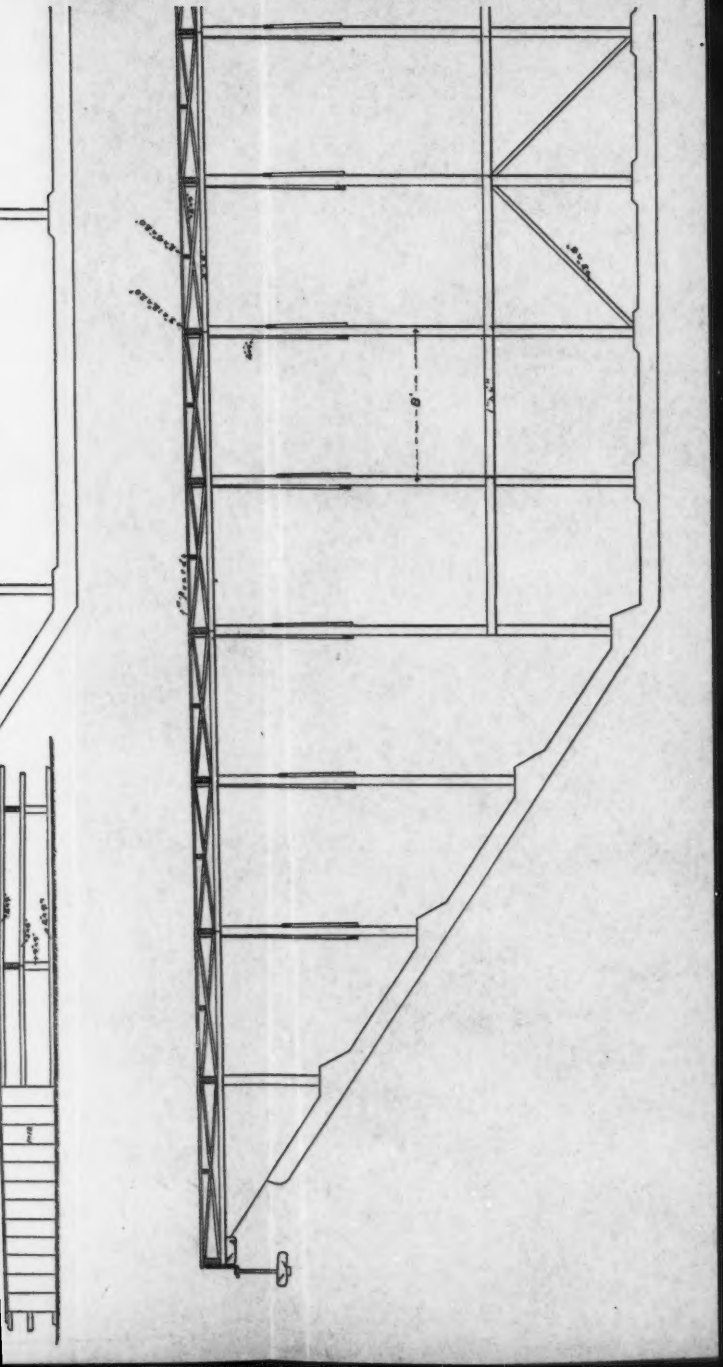
SCHUYLER ON WATER WORKS OF DENVER.



every space, this was, perhaps, a needless precaution. A valve well or shaft was sunk in the summit of the rim of the basin, 900 ft. from the inner portal of the tunnel, to intersect the tunnel. It is 12 ft. in diameter, lined with stone masonry 2 ft. thick, and was carried up above the surface to the level of the top of the dike when finished. The dike will be built around this shaft. A second tunnel, 25 ft. above the lower one, and of the same size, was run from the interior as far as the gate shaft, 600 ft. This tunnel is expected to be the one principally used, as it has nearly 90% of the total reservoir capacity above it and will draw water nearer the surface, where the quality is generally better than at the bottom. Moreover, the water from this tunnel, on reaching the valve shaft, will have a vertical drop of 25 ft. to the level of the outlet, and will thus be well aerated. The shaft is carried down 10 ft. below the outlet tunnel, forming a sump to receive the shock of the falling water. Gratings of iron across the shaft will also serve to break the water into spray and mingle it more thoroughly with air. Pressure valves, to be operated from the top of the shaft, are placed at each of the tunnels on the lake side, and a double set of perforated plate screens extend from top to bottom across the shaft.

The inlet valves, at the inner portals of the tunnels, are novel in their construction and operation (see Plate XIX). A railroad track, with 30-lb. T rails, and cast-iron cross-ties resting at each end on concrete piers 12 to 18 ins. above the ground, is laid from the high-water level down the slope to the mouth of the lower tunnel, crossing on the way the mouth of the upper tunnel. An oval valve seat, 4 ft. long, 2 ft. wide, of cast iron, imbedded in concrete, forms the portal of each tunnel. The lower one is set at an angle of 45°, the upper one on the angle of the slope, about 18° from the horizon. The valves to cover these openings are hung on a low four-wheeled car suspended by hangers, which, standing on the slope, swing the valve off the center sufficiently to clear the track. On reaching the seat, a stop arrests the valve, while the car moves on until the hangers are perpendicular to the frame of the car, which allows the valve to drop to its position. The car is raised and lowered by a cable that rests in loose pulleys with deep flanges, placed at intervals between the tracks. When the valve is to be raised, it can be brought to the surface and a screen lowered in its place. The device is simple and operates easily. It was preferred to a masonry tower standing out in the lake where it would

be exposed to severe ice thrust, and might at times be very difficult of access.

A second outlet to the lake was provided on the north side, by means of a tunnel 500 ft. long, at the same level as the upper tunnel of the main outlet, and a gate shaft of the same diameter as the one described, and similarly arranged, was erected. This is designed for future connection with a direct pipe line projected to the Ashland Avenue reservoirs. Both shafts are provided with inlets at the level of the present ground surface, which will draw water 20 ft. from the top when the dikes are completed and the lake is full.

The Third Conduit.—The water-shed of Marston Lake is not over three square miles in area, and the lake has no means of being filled except by the construction of feeders to it. To supply the reservoir, a conduit of 60 000 000 galls. daily capacity is projected to be built from the Platte River. This conduit will be 11½ miles long, of which nearly 3 miles will be in tunnels, 10 in number, through earth or soft sandstone. Wood pipe will be used for the remainder. Pending the completion of this conduit, a temporary arrangement for filling the lake has been effected for the use of an irrigation canal, leading from the cañon of Bear Creek, an important tributary of the Platte. A flume, 1½ miles in length, with a capacity of 80 cu. ft. per second, has been built from the canal to the lake.

The waters of the Platte River are so entirely appropriated for irrigation during seven months of the year, that, in order to maintain a constant supply flowing into Marston Lake, a mountain reservoir on the head waters of the stream has been determined upon as a necessary part of the water works, to store the surplus flood water of early summer and the unused flow of winter. A number of sites of large capacity are available, and active work upon a masonry dam of generous proportions will be one of the next items of construction to be undertaken.

When completed, as it has been outlined in these pages, the system will be one of the most perfect in the United States, for amplitude of supply for a much larger population than the city now has, and for purity, clearness and general excellence of quality. The two conduits already built have a combined capacity of 24 000 000 galls. daily from the river, or 30 000 000 galls. if part of the supply be drawn from the higher level of Marston Lake Reservoir, while the extension of the

LAND'S

Railroad Gate.

RECONSTRUCTED BY

Chas P. Allen.

Chief Engr. Citizens Water Co

DENVER, COLO.

Scale ½ = 1'

third conduit from Marston Lake to the Ashland Avenue reservoirs, will have a capacity of 20 000 000 galls. daily. As the latter is but nine miles long and can be built in a few months whenever needed, it is evident that no danger of water famine need ever threaten the city, after the main feeder to the lake is completed.

The present consumption of water in Denver is enormous, reaching fully 200 galls. daily per capita in the summer and fall months. This is, doubtless, largely due to the reckless waste stimulated by "free water," which the exigencies of sharp competition between the two companies have enabled the citizens to enjoy for two years. The normal consumption, under ordinary prudential use, even in a climate as arid as that of Colorado, will probably not exceed 125 galls. per capita. The present population of Denver is about 150 000, and the summer consumption is estimated at 30 000 000 galls. per day. Even if this rate be continued and increased, the works of The Citizens' Water Company, completed and under construction, will amply provide for the needs of the growing city. The entire plant, when completed to the extent now contemplated, will represent an outlay of about $3 000 000. The company has no indebtedness, no interest charges to meet and small operating expenses. Its rival has an outstanding debt of $4 200 000, and is in the hands of a receiver. Under these conditions, the consolidation of the two companies may be shortly anticipated, and the novelty of "free water with a chromo" brought to an end.

DISCUSSION.

R. C. Gemmell, Assoc. M. Am. Soc. C. E. (by letter).—It may not be out of order to discuss a few of the practical details of construction of wood stave pipe, such as described by Mr. Schuyler. In view of the rapidly increasing use of this pipe and the widely varying opinions in regard to the best methods of construction, the subject certainly is an important one. On the Pacific slope, at least, where such excellent lumber is so plentiful and cheap, stave pipe will, undoubtedly, to a great extent, be used instead of iron pipe for water-works conduits, and instead of box flumes on irrigation systems and water-power plants. It is not because I feel that I am particularly competent to discuss the subject that I bring it up; on the contrary, it is simply with a desire

to provoke a discussion that will bring out all the knowledge and experience of other members of the Society bearing in any way upon the points mentioned below.

In regard to the material used in making the staves: First, should it be wholly free from knots? and second, should it be thoroughly seasoned? It is claimed by some makers that small knots, where they extend through and not across the stick, do not occur oftener than one knot in every 2 ft. in length of stave, and none within 1 ft. of the end of the stave, are not so objectionable as to justify the expense of requiring perfectly clear stuff. If the pipe is to be subjected to considerable pressure, however, it would seem to be good economy to use clear lumber. As to seasoning, Mr. Schuyler states that "lumber must be thoroughly dry and seasoned before milling, and after milling the sooner it is built into the pipe the better." For redwood this may be all right. Redwood, in addition to being very soft, possesses the property of shrinking (or swelling, as the case may be) very slightly transversely and very considerably longitudinally. By using it well seasoned, then, there may be no particular danger of setting up any great strain in the bands after the water is turned on, and, perhaps, the longitudinal expansion would only serve to make the butt joints all the tighter. For some work in which red fir is to be used, however, the requirement is that the lumber shall be cut, milled and built into the pipe as quickly as possible. Experience shows that the shrinkage of red fir from a green state to even a partial state of seasoning is fully 4%; hence, it is argued that the only safety in using it without special appliances for taking up the swelling is in putting it in green. Exception may be taken to this by some, however, by reason of the fact that green red fir seasons and shrinks to a certain extent even under water, thus, perhaps, causing loose joints and leakage in the pipe; and also, because of the danger of the sap in the wood, causing decay. With the pipe under considerable pressure, perhaps neither of these objections would hold, as it might be said that the water would force out the sap and penetrate the pores of the wood so thoroughly as to effectually prevent any shrinkage.

For preserving the wood an outside coating of hot asphaltum for the staves is advocated by some, while others think it unnecessary and therefore a needless expense. Those in favor of it claim that it makes an impervious coating, which preserves the surface of the staves and prevents the evaporation of the water that penetrates the pores of the wood from the inside. Those against it claim that the water itself, by completely saturating the wood, is a sufficient preservative, and that the floating of the staves in a bath of asphaltum is sure to leave some asphaltum on the radial edges and the ends, making it difficult to get tight joints; while if the coating is done with a brush, it is very hard to make the asphaltum stick to the wood. This difference gives rise to

another difference of opinion, namely, as to what constitutes the proper thickness for the staves. It is admitted by both sides that the danger of decay is confined to the outside of the pipe, where, under light pressures, the question of saturation may not be without uncertainty for a thick stave. Starting with a safe deflection for a thick new stave, if decay does take place, there may be eventually an excessive deflection, to say nothing of the leakage resulting from the loosening of the bands. On the other hand, the velocity of the water in most cases is high, and if the staves are thin when first put in and the water not clear, there may be so much wear on the inside as to at last cause trouble. My own judgment would be to thoroughly coat the staves on the outside with hot asphaltum, put on with a brush; and to make the staves thick enough to absolutely prevent any appreciable deflection, when using bands of the most economical sizes, spaced for a factor of safety of four.

In making the butt joints sometimes wooden splines are used, instead of the metallic tongues described by Mr. Schuyler. Each method has, of course, its advocates. Those in favor of the metallic tongues claim that in using the splines it is hard to made a good fit, while those in favor of the splines claim that the acid in fir wood will corrode the metal tongues, causing rust and decay. Probably either method makes a good joint. Some builders even go so far as to claim that neither the tongues nor splines are necessary in order to made tight joints. By some it is required that the saddles shall not be closer than a certain distance to butt joints, while others think it is of no importance how the saddles are placed.

The statement that "it is desirable to tighten the bands until they are imbedded in the wood" probably refers to cases in which the material used for making the staves is redwood or some other very soft wood. In order that the seams may be tight, it is necessary that the staves should press against each other with a pressure per square inch at least somewhat in excess of the pressure of the water. This point is an important one, and the cinching can probably be properly done only by skilled and experienced workmen. Perhaps a good method would be not to cinch the bands up very tight at first, turn the water on and allow the wood to swell, and then adjust the tension on the bands properly. That the joints may be tight, and still that no appreciable strain may be put on the bands on account of the swelling of the staves, "is a consummation devotedly to be wished," and worthy of the most careful consideration. In the use of comparatively hard unyielding timber in a seasoned condition, and subject to a lateral expansion of several per cent. while in the process of saturating, it would appear that, with the bands tightly drawn at the time of construction, such swelling might strain the bands beyond the limit of their elasticity. That such danger is recognized by some

engineers engaged in the building of pipe of this character is evident from one having sought a solution of the difficulty by inserting a number of corrugations in the band. It would seem that this effort to obviate the difficulty would require very careful adjustment of the corrugations, and some practical experimenting, to determine just what character of corrugation would remain wholly unyielding under the working strain, and yet yield sufficiently to take up the swelling well before the limit of elasticity of the band was reached. Again, with strains existing in the bands just sufficient to cause a yielding in the corrugations, would not any additional and unusual increase in the water pressure produce an increased permanent yielding, leaving the bands more or less loose after the unusual pressure had subsided, and thereby causing leakage in the pipe?

Mr. Schuyler is certainly very conservative in estimating the life of the pipe at "from 15 to 20 years," as with the bands and saddles properly coated it would not seem unreasonable, in the light of past experience, to expect the pipe to last fully twice as long. Some builders will also be likely to take exception to the statement that "the bands are the only perishable portion." Mr. B. H. Hull is on record as having stated that the pipe "wears out on the inside," the rapidity of such wear depending to a great extent upon the condition of the water, whether clear or laden with silt or grit.

A question frequently asked is: "What is the maximum pressure under which this pipe can be used?" Generally the answer is that the bands can be proportioned and spaced to stand almost any pressure, and that all depends upon the hardness and strength of the lumber. The prevailing idea among engineers seems to be that, for a safe limit, a pressure of a little less than 100 lbs. per square inch should not be exceeded. The probability is that this pipe can be safely used to all pressures under which it can successfully compete with iron pipe in the matter of first cost.

The above points regarding seasoning, coating, jointing and cinching, as well as those of the probable life of the pipe and maximum pressure to which it may be subjected, have been mentioned because there seems to be such a variety of opinions in regard to them. At least two members of the Society are interested in the manufacture of wood stave pipe. It is to be hoped that they, and any others who may have had occasion to give the matter study, or may have had actual experience in the construction of the pipe, will give us the benefit of their knowledge in relation to mooted points.

A. Fteley, M. Am. Soc. C. E.—In my personal experience I have had more to do with iron pipes, and the question of cast-iron *vs.* wrought-iron or steel pipes has been more conspicuous in this part of the country. My knowledge of wooden pipes for water supplies of cities and towns is almost entirely limited to those of small diameter, which

have been, and, I understand, are still being, made of perforated logs, sometimes strengthened with thin spiral iron wrappings. The long life of these pipes, without any additional supports, is attested by the samples in good preservation, which are, even now, dug out from the streets of New York, and which were a part of its early supply system. It is well known, also, that in our neighborhood penstocks have been made successfully in many mills of wooden staves hooped with iron. It would be consequently difficult for me to speak knowingly of the merits of the pipes just described, but I do not doubt that in localities where proper timber can be found in abundance, and where transportation is expensive, they can be profitably substituted for iron pipes.

Mr. Schuyler's paper shows the prominent fact that an unusual number of successive water-works schemes have been built in Denver. That the first system should have been established on a small scale at a time when nobody could predict the large development of the city, is easily understood; but as soon as the rate of increase of the population, and the establishment of local industries, indicated the almost certain growth of the community, it would have been much wiser to provide for the future, under good engineering advice, such system as would have been susceptible of progressive development, thus securing the best result at the least cost.

This means of foresight has been common to a large number of communities, and shows once more the advantage of proceeding from the beginning in a more rational way.

This paper recalls also the fact that many people are in utter ignorance of the capabilities of water sources. When water is to be collected from the surface of the ground, the capabilities of the watershed to yield water are generally well understood; but when it is a question of underground sources, the apparent mystery attached to the matter sometimes induces the wildest speculations. From a comparison with such works as are now in existence, and from the local circumstances of each case, a reasonable estimate of the capacity of the source in hand can reasonably be made. In places where the amount of water required is small, and where the collecting and storing grounds are extensive, the loss of storage in the underground reservoir from year to year may appear so small as not to be sufficiently noted, and the conclusion may be hastily drawn that the source of supply is practically inexhaustible; but we know by experience that after a time the loss of storage is likely to be felt disastrously, especially when (as is almost invariably the case) the draft from the underground sources increases gradually. To illustrate the extravagant expectations sometimes entertained by communities as to the results to be obtained at a comparatively small cost from underground sources, I may give an instance where a supply of 100 000 000 galls. per day was confidently ex-

pected by a considerable portion of an important population, from underground sources, while an impartial survey indicated a possible maximum of 5 000 000 galls.

JAMES OWEN, M. Am. Soc. C. E.—I do not know, Mr. Chairman, that I have had any experience in wooden pipes, but I have had some little groping underground for underground water. There is one curious point about underground supplies that struck me. In the district where I have my practice the water is almost entirely from the red sandstone, and the uniformity of its delivery is, to a certain extent, remarkable. An ordinary driven bored well, 4 ins. in diameter, carried down to 70 or 100 ft., will deliver 30 000 galls. a day. On visiting, some years ago, the Brooklyn Water Works, and asking the engineer, Mr. Van Buren, what he estimated as the delivery of each of his driven wells in the sand of Long Island, he gave me the same figures, that he estimated the delivery of each of those wells nearly at 30 000 galls. a day; it is a curious coincidence.

There is one point that I wish to allude to in this question of underground supply that has not been much touched upon; that is, in reference to the normal delivery of a well from which the water is continuously taken. At the first instance of pumping, the delivery, of course, is much greater than may be expected in the future. As time goes on and the pumping is continued, there is a probability that the permanent delivery may be more than the apparent delivery for the first few years, from the fact that you are creating a current through the soil for the rain water, and although after three or four years the wells may show a remarkable lessening of capacity, yet that may not be the true capacity. The instance I know best was the well put down in Brooklyn Park in 1868, where the first delivery amounted to about 670 000 galls. a day; by pumping for a few years that delivery was reduced to about 400 000 galls. a day, which came to be its normal delivery. I have not seen lately what the statistics show of the delivery, but I think it is fair to presume that it will never be less than that.

In my experience I noted one peculiar thing: I have a plant in which I am using eight wells in the sandstone, and in dry weather the shortness of the delivery becomes apparent with the lack of rainfall, but after a rainfall there was no perceptible increase of flow in the wells until the third day, and on the third day after the storm we always began to appreciate the increase of flow due to the preceding rainfall. It gave me an idea of the velocity with which the surface water would be carried down through the overlying soil, about 20 ft. in depth, until it arrived at the rock and flowed out there.

I would state another instance of underground delivery that occurred in the city of Newark some 20 years ago. When the original plan of the water works was designed, it was proposed to construct

two large filtering beds alongside the river ; they were constructed for that purpose, and they are there to-day. The idea was not to take the river water, but to take the underground water adjacent to the river and pump that for the delivery to the city. The works were built and pumps started, to deliver that water alone. The capacity grew very inadequate for the requirements of the city, and the gates were quietly opened from these chambers, and the river water and the spring water were used together. Lapse of time led people to forget, partly, the ideas that were in vogue then, and the existence of these large beds were partly forgotten, and about 10 or 12 years after that there came to be a craze for the driven well system, and they worked up a sentiment in favor of the driven wells, and the driven-well people put a plant alongside the original filter beds, with the idea that they would be able to deliver 20 000 000 or 30 000 000 galls. a day to the city. Pipes were put down and pumps connected with them, and the amount taken from the driven wells was exactly the amount furnished by the intercepting wells.

I would state that the area tributary to the wells was about one and one-half miles long, with an average of three-fourths wide; to the south it was entirely intercepted by a high ridge, which prevented the water from flowing that way; on the west was a trap-rock ridge, and on the east and north by another water-shed entirely.

L. L. Buck, M. Am. Soc. C. E.—What was the depth of these wells ?

Mr. Owen.—The average was about 100 ft.; we had one large well which ran down 50 ft., and three diamond drill borings went 50 ft. farther. One went down 107 ft.; that was in red sandstone.

That was entirely through gravel. The sandstone has the lines of cleavage and lines of stratification. The water will percolate much more freely than through sand.

Edward P. North, M. Am. Soc. C. E.—Mr. President, I had in Mexico a little experience with driven wells which was new to me, and I have not seen anybody that has had a like experience. On a very flat shore, which was mostly clay, there were some sand dunes, and in the center of these dunes was a depression which always contained fresh water, the surface of which was only 1 ft. or so above the level of the salt water. The sand dunes came down to the seashore, and I concluded that they had acted as a condenser of the dew and the moisture contained in the air, and the fresh water was lying on top of the salt water because it was lighter. I drove wells, and found I could get fresh water about 6 ins. above high-tide level; but after we had pumped for the use of thirty men for about two days, the well would get brackish; it would be necessary to move the well some 20 or 30 ft., when we would get fairly fresh water for some two days more. The well was moved around in this way, and that was the main source of supply for some

three or four months, saving us bringing water some two miles. There was one other place on the shore where fresh water was obtained from a sand dune, that had on one side of it salt water and on the other side a salt marsh; whether the fresh water would be exhausted by a little pumping I do not know; it probably would have been.

E. E. Olcott, M. Am. Soc. S. E.—Seeing that Mr. North takes up Mexican experiences, I am rather at home. I had an experience at one time in striking water in a dry cañon, to the wonderment of the Mexicans, where quite a large supply was needed. It was a mooted point for some time as to where the supply had best come from, and by studying the direction of the bed-rock at the side of the valley, which was entirely dry for the greater portion of the year, I judged that we could strike water in the sands beneath. By sinking a well 35 ft. deep and running drifts out across the channel, we did strike a very large stream of water which was successfully pumped up through 1½ miles of pipe to an elevation of 575 ft., the pumps used being duplex compound Worthingtons.

In regard to the wooden stave pipe, I consider that the promise for its successful introduction is very good in the West, especially in California; it is used already in connection with irrigation enterprises. I think that if it had been introduced during the time of hydraulic mining in California it would have been largely employed, as it offers advantages over iron pipe for bringing water to the hydraulic claims. The abundance of splendid timber there would have made it very cheap. An advantage this wooden pipe offers in some work is the facility with which it can be tapered and reduced from one size to another and connections made.

I was surprised to see the statement that the percolation through sand was not much more rapid than from ½ mile to 2 miles per annum.

I was interested in another point in the paper, viz., the scraping out the sediment from the reservoir. I saw an interesting case in Colombia, S. A., where the healthfulness of a locality was destroyed by flooding a large basin without first clearing away the vegetable matter. That is something that engineers are not careful enough about; even the vegetable mold should be entirely scraped up and removed as thoroughly as possible.

L. L. Tribus, Assoc. M. Am. Soc. C. E.—I can, perhaps, throw a little additional light on two points in this subject. In the southern portion of New Jersey there are a good many driven wells of about 50 ft. in depth. The rainfall is rarely felt in them under about 30 hours, while it takes a very heavy storm to show inside of 24 hours.

This indicates a vertical filtration of about 3¼ miles per annum.

As to the other point, the durability of wooden pipe; a few years ago I had occasion to take out wooden water pipe that had lain in the ground for nearly 40 years, part of that time used in the water supply,

part of the time dry, the ground itself, sometimes wet, sometimes dry, and yet the logs were in perfect preservation. They seemed to be yellow pine.

Mr. NORTH.—What soil was that ?

Mr. TRIBUS.—In sandy soil, near Mobile, Ala.

Mr. FTELEY.—Mr. Chairman, it seems to me that Mr. Gleim could give us some interesting comments on the subject, as he is connected with the city of Hamburg, and we have all heard of the works for pipe-laying in that city. We would be very glad to hear from him.

C. O. GLEIM, Cor. M. Am. Soc. C. E.—I am not prepared to speak on the subject of, and, not having anything to do myself directly with, those works, I am only half familiar with them. I do not see exactly that it has a great deal to do with the subject under discussion this evening, except that it is water works. The city of Hamburg has drawn its supply from the River Elbe ever since the time of the great fire, about 51 years ago. The water works had been planned at the time the city was burned down, and the works were commenced immediately. Originally they were planned with a view to guard against another case of fire ; it was not at that time meant as a source of supply for house use, but it was afterwards also used for that purpose. The water from the river was let into large subsiding reservoirs, and from them drawn into the city. But from a very early time it had been contemplated to introduce filter basins so as to improve the quality of the water for potable purposes. The introduction of the filtration, however, has been delayed through various causes, for a long time. There was at one time a scheme brought up by one of the members of the Common Council, a doctor of medicine, to introduce what he called a peripheric system of filtration ; that is to say, not to have one central system, but to have local systems of filtration. It was because of the effort in that direction that the scheme for central filtration was delayed. Another scheme was to draw water from lakes in the Duchy of Holstein in the neighborhood, which, however, was found to be inadequate ; either it did not furnish a sufficient amount of water, or the water was of inferior quality; and if it had been satisfactory, there were difficulties in obtaining the water, owing to the political boundaries. So, after all these delays, the authorities at last voted the money to build the great central filtration scheme, which, by the scale of expenses that we are used to in Germany, was quite an expensive one. The amount of the first appropriation was 7 000 000 marks, and that has been entirely exhausted, requiring some additional appropriations.

The old intake for the water works was about four miles above the inlet of the effluent of our sewerage, which empties into the River Elbe below the city. As you all know, the river is subject to ebb and flood, and the tide current takes the polluted water from this effluent

of the sewage up to the river, but only to a point some distance below where the intake used to be. In the present filtration scheme that intake has been taken still further up the river, about 1½ miles I should say. We have now an excellent supply. The works were two-thirds finished when the cholera came last year; it came one year too early, or it would have found us better prepared. The water was very poor before this filtration scheme was carried out. It would have been of better quality if we had not thought of filtration, because the city would have had to enlarge the settling reservoirs; but with this scheme in view, we have never spent any money in this way. Since last June we have the filtered water, and from the reports that I have seen in the newspapers, the few cases of cholera that are now there are attributed to remnants of the poor old water in the pipe system and probably to the incrustations of the old pipes, which will require a certain time to get rid of; but I believe that with the present system the city will not be subjected to any such large epidemic as we have had last year.

J. D. FOUQUET, M. Am. Soc. C. E.—I might give some information as to the durability of wooden flumes, made after the barrel type, as I have made some investigation on the subject. At Wicopee on Fishkill Creek are located the New York Rubber Company's mills, and just above the works is a dam furnishing water-power. Just below the rubber works is located a grinding mill, erected many years ago by John and William Rothery, manufacturers of files. Power is brought to the grinding mill through 300 ft. of a wooden cylindrical trunk, 2½ ft. in diameter; staves were made of the best clear white pine, 5 ins. wide, and about 1¾ ins. in thickness. Iron hoops are arranged about 2 ft. apart. The trunk is supported on wooden trestle bents, which have been repaired from time to time. The trunk which was built in 1850, Mr. Rothery assures me, is in good condition at the present time, and is good for ten years more of service.

Another shorter trunk at Carrol's straw hat works, about 1½ miles above this, is partly under and partly above ground; was built about twenty years ago, and is now in excellent condition.

Hoops are arranged to cover the joints of staves, which are spaced to suit the length of the stave plank.

R. L. HARRIS, M. Am. Soc. C. E.—As the durability of wooden water pipes seems to be an interesting point this evening, I would say that the city of Portsmouth, N. H., has good examples.

The original water pipes were bored pine "pump logs," joined by taper point and socket.

Some of these pine logs of 5-in. bore, laid so early as 1797, were removed in 1870. These were found to be sound, but were worn on the inside to diameters of about 8 ins., as was well shown by the smoothly polished knot wood projecting to the inside, the softer wood having

been worn away by the water. The location of these logs was in a main one mile out of town, through wet meadow-land.

In this connection the following may be of interest: Mr. Oliver Ayres was Superintendent of Portsmouth Aqueduct for 50 years previous to 1884. Mr. Joseph Akerman was superintendent for 30 years before Mr. Ayres. Mr. Akerman told Mr. Ayres that he had never seen the service pipe to the colonial mansion of Governor Langdon. This service pipe, a pine pump log, was uncovered after 1875, by reason of a stoppage; it was found to be sound, a root seeking water had grown through a joint and had blocked the bore; this pipe was in wet clay about 5½ ft. beneath the surface. Here, then, are two examples of sound pine log pipes of over 70 years of age, both a main and a service pipe.

In gravely hills the pipes were replaced when 30 to 40 years old.

As to the matter of removing vegetation, we are all familiar with the fact that it is necessary to remove this, but a special case has come under my observation this summer where a city of 40 000 people built water works two years ago and are now paying for drinking water 5 cents a gallon, the reason being that the vegetable matter was not removed from the reservoir when it was built.

J. D. Schuyler, M. Am. Soc. C. E. (by letter).—As Mr. North's successor in Mexico I had occasion to make a study of the sand-dune water supply to which he refers, with the view of utilizing it to supply locomotives at the western terminus of the Sinaloa and Durango Railroad, and came to the conclusion that the source of supply was entirely independent of the rainfall and was probably due to condensation of moisture from the air which penetrated the sand far enough to reach a much lower temperature. The supply was constant and reliable, but limited in volume, and was apparently in direct ratio to the size of the sand pile. A similar phenomenon is observable in San Francisco, where the sand dunes are piled up to a much greater height and yield larger volumes of water. The first water works of San Francisco derived their supply from one of these little sand-dune lakes, near the Marine Hospital, and so near the sea level that the flume carrying the water to the city skirts the shore all the way just above tide water. This is still in service. Laguna Honda, from which several million gallons daily are pumped to the city, is another sand-dune lake near the ocean level and separated from it by a narrow sand spit. Neither of these lakes has a water-shed large enough to account for the supply which is derived from them as possibly coming from that source, and I have no doubt that they are perpetually replenished by condensation of moisture in the atmosphere.

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TRANSACTIONS.

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TRAIN LOADINGS FOR RAILROAD BRIDGES.

By THEODORE COOPER, M. Am. Soc. C. E.

READ JANUARY 3D, 1894.

WITH DISCUSSION.

The great advance made within recent years in the matter of metallic bridge construction, and the gradual convergence of opinions towards uniformity in our practice, has led many to hope for and believe in a final solution of all the various minor points connected therewith. The writer never has accepted any such idea, and does not believe that we have reached the end of our education or development in any direction. The tendency to seek for a "final standard" in any engineering construction, regardless of changing conditions, is a harmful one. Take from the engineer the right of professional judgment, and how does he differ from an automaton? While the writer does not believe that all the disputed points in engineering practice can ever be brought to positive uniformity, we can, by fair and thoughtful discussion, approach it more and more. To do this in the most effectual

manner, it will be better to limit the discussion so far as is practicable to one subject-matter at a time.

The one subject upon which there is more general interest than any other connected with railroad bridges is the character of the live loading to be specified, and upon which there is a far greater probability of harmony than many others.

Which mode of obtaining the strains, due to the finally selected loading, is adopted, is a matter of minor importance, provided the results are close approximations to the true strains produced by the standard loading; for we must, for fair comparisons between bidders, have some standard of final reference in case of difference, and there is none that can be generally selected which would be simpler or more positive than an actual train load.

Throwing aside, therefore, all references to relative merits of actual strains, diagrams, tables or other methods of rapidly approximating to the truth, let us take up and simplify the train-loads by reducing them to their simplest forms and least number to cover the necessities of various classes of railroads.

The miscellaneous collection of train diagrams now in use has grown tentatively and naturally till it has reached its present absurd condition. The first engines selected were properly the heaviest actually existing engines, with the useless refinements of all their fractional dimensions and weights. In no other way could the managers of our roads have been made to pass from the uniform loads then in general use to a higher loading for their bridges. No theoretical or badly proportioned engine would have been accepted. Such would have been considered as evidence of the ignorance or impracticability of the engineer, not only as to the proper proportions of a real engine, but as to the desirability of a more suitable train load by which to proportion their bridges.

That it was the most important advance towards better bridges will not be questioned by any one having knowledge of the past history of railroad bridge building. As this step forward was necessary upon all our various railroad systems, it was natural that each engineer selected the engine and train best suited to meet the circumstances of his own case. The next step was the inventing of typical engines to represent the expected developments in the future traffic of the various roads. The result is naturally one of confusion; but, nevertheless, it

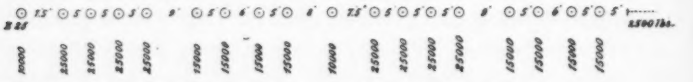
was a necessary evil for the time being. The ultimate purpose has now been accomplished and we can now readily reduce order out of chaos by the selection of a few typical train loads sufficient to cover all practical cases.

We can all agree upon the uselessness of refinements in dimensions or weights.

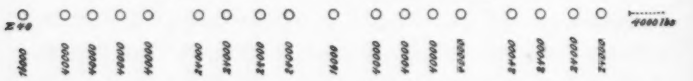
It would also seem possible to agree upon two types of engines to represent the loadings produced by passenger and freight trains with varying wheel loads to suit each special case.

We can also accept, without question, the freight train loads as the only car loadings necessary to consider, for the heaviest passenger car (114 000 lbs.), crowded with people, will not give a load per lineal foot of track over 2 000 lbs., nor a wheel load exceeding 12 000 lbs.

If the above general statements can be accepted, it would appear to the writer possible to reach a harmonious conclusion as to the standard train loads suitable for different railroads, and that such train loads can be arranged so as to be changeable by regular gradations to any extent, thus meeting all present conditions and providing for any future ones, without introduction of any complications in methods of computation other than uniform increase by percentages. Then each computor, who has his special method of determining the strains due to any train loading, whether by tables, diagrams or otherwise, can readily change to any new condition by increasing all the strains by a fixed proportion. For example, if we assumed the following diagram as representing the standard freight engine and train for minimum loading:



we can pass by regular gradations to the Lehigh engine or



to cover different conditions, without departing widely from the proportions of actual engines.

As the loadings throughout the whole train are all fixed proportionally to the governing load on the drivers, the strains in each case will also be directly proportional to this load; therefore, all strains due to

any engine and loading of the series can be obtained proportionally from the strains due to Engine 25.

The foregoing diagram for the passenger engine can be increased in like manner. If, then, we label our minimum train-loading E. 25, we can adopt a nomenclature readily understood by any one; as, for example, E. 30 would be a train load 20% heavier than E. 25, E. 35 40% heavier, etc. The live loads in the two cases being in like manner 20 and 40% heavier than those for E. 25.*

The previous diagrams are only offered at present as illustrative of the method proposed by the writer. If that be accepted as a desirable one, the exact values to be selected for the several weights forming the minimum train loading can be more readily reached by mutual discussion.

As a preparatory step towards this discussion, the writer has collated certain data to be attached to this article as bearing upon the possible future train loads.

The great increase in the weight of engines and car loads during the past few years has impressed many with the belief that the increase for the future may be at a similar rate. This is hardly possible, as we shall endeavor to show. There are several factors that tend to limit the increase to a possible determinate maximum, as far as the load per lineal foot of track is concerned, and this is the principal point of train loadings. These factors are the limitations of the cross-section of the car-body, the capacity of the axles or journals, and the methods of loading the cars.

In like manner, the advance in weight of the engines is controlled by the maximum load on one driver and numerous constructive features of the general machine. As the engine is practically limited to one line of railroad and often to a single division of such railroad, the restrictions are not so general as for cars which are liable to pass over wide ranges. The peculiar features of special roads, such as the roadbed, rails, curves and so forth, may allow the use of engines totally inadmissible upon other systems. But that there is an economical limit to the size of our engines is certain, for, not only can we see

*The writer has had occasion recently to use this percentage method, being asked by the general manager of a railroad to specify an engine and train-loading considerably heavier than his class, heavy A, but not as heavy as the Lehigh; the live load was changed to one, 20% greater than heavy A, or the strains produced by heavy A were increased 20 per cent.

a limitation to the engine wheel loads in spite of improved roadbeds and heavier rails, but also a final compromise in the existing struggle between the tractive power of the engine and the draft mechanism of the cars. The destruction of freight trains on most of our mountain divisions through the breaking away of the train, from failure of the draft mechanism to equal the hauling capacity of the engines, must wipe out an important part, if not all, of the gain hoped for by the use of much heavier engines.

As for the cars, these, by the necessities of ready exchange from road to road without transfer of freight, must be adapted to the general limitations of the general railroad system of the continent; hence, there are limitations to the width and height of such cars. The width of cars cannot pass certain accepted limits without affecting the clearances at existing bridges, tunnels, freight houses, station platforms, etc., of all roads; and for double-track roads it would require increased width between tracks, involving wider cuts, embankments, tunnels, bridges and other changes. Increased height, also, would involve changes in tunnels, bridges, etc.

We can therefore assume that radical changes in the width and height of cars are not very probable from the present outlook of clearances. Moreover a careful consideration of the fact that we are not now using the full cross-section of our car-bodies for more than a small percentage of the freight handled by the railroads, on account of the great loads that would be imposed upon the journals and the greatly increased strength of frame needed, is an additional reason for believing that wider and higher cars are not probable.

The cross-section of the average box car in general use does not exceed 55 to 60 sq. ft.

The largest car-body known to the writer is the 40-ft. furniture car of the Chicago, Burlington and Quincy Railroad (see *Railroad Gazette*, January 27th, 1893). This has an inside cross-section of 74 sq. ft.; but only 3 ft. and 4 ins. in depth is allowed to be used for wheat and corn (capacity limited to 50 000 lbs.), its full inside height being 8 ft. 9 ins.

If this car were built strong enough to be filled with wheat or corn, retaining the same outside dimensions, the useful cross-section would not be more than 70 sq. ft.

Such a car fully loaded with corn or wheat would have a capacity of 130 000 lbs., far in excess of any car now in use; it would, however,

only impose a load of 4 000 lbs. per foot of track, allowing 1 000 lbs. per foot for the weight of the car itself.

In Table No. 1 the writer has collected the general dimensions of the heavier cars in actual use to-day upon American railroads, with the loads they impose upon the track per lineal foot and upon each axle. It will be seen that, with the exception of the gun cars, no cars are intended to impose a load as high as 4 000 lbs. per foot of track, or more than about 24 000 lbs. upon a single axle.

Table No. 2 gives a summary of the freight handled by a number of the most important roads (many roads do not give any classification of the freight handled by them), which we may accept as an approximate guide in consideration of this subject.

In Table No. 3 is given the weight per cubic foot of the principal articles other than ores and metals, measured on the space occupied by such articles as usually stored.

In Table No. 4 are gathered the weights per square foot of floor space of a number of representative classes of machines. A careful study of these tables appears to the writer to justify the following line of reasoning:

The average box car, which may be taken as best representing the car of greatest storage capacity, has about 33 ft. of effective storage length, and occupies about 37 ft. of track. With a possible maximum cross-section of 70 sq. ft., the storage capacity of such a car would be 2 310 cu. ft.

If this space could be fully utilized for the heavier coals (56 lbs.), the carrying capacity of the car would be 130 000 lbs. and the load per running foot of track (allowing 1 000 lbs. per foot for weight of the car) would be 4 500 lbs.

Loaded with corn or wheat, the capacity would be 111 000 lbs. and the load on the track would be 4 000 lbs. per foot.

It would be impossible to exceed these loads by a large part of the "machinery," "castings," "structural iron-work" and "stone," on account of the difficulties of storing them on the cars in a compact shape.

If, therefore, we consider the difficulties of loading and fully storing such cars, it would appear that 4 000 lbs. per foot of track will cover at least 90% of the freight handled by the general railroad system of the country.

Such a car as we have assumed would weigh, loaded, 148 000 lbs. This load would give 37 000 lbs. on each axle for four axles, or 24 666 lbs. for six axles, which would require two 6-wheeled trucks under each car, or three 4-wheeled trucks. While this last method has been employed for certain ore cars, it is hardly likely to become the practice.

In addition to the above facts, it must be borne in mind that cars built for these heavier loads must be made stronger and necessarily heavier; that such heavier cars must frequently be used for the larger body of lighter freight handled, in order to keep up a paying general circulation of rolling stock, and that in such cases we have increased the non-paying tonnage to the detriment of a large body of freight to benefit another class. The natural tendency must therefore be to get cars which will be the best suited for the average rather than the heavier freight.

All things considered, therefore, we think it very improbable that our train loads will ever exceed, as a rule, 4 000 lbs. per foot of track.

No doubt we can also agree that we ought to construct the bridges of even minor or weaker roads up to a standard equal to the diagram E. 25, or can start with this as a minimum loading.

As passenger engines are liable to pass over even minor railroad systems, which may not have a heavy freight traffic, to make a connecting line for some general through passenger route, bridges of all roads should be proportioned for the heavy passenger engine.

While passenger trains are lighter than freight trains per running foot of track, the concentrated loads on the passenger engine frequently induce heavier strains upon the shorter spans or panels than the heavier freight engines.

In addition, therefore, to the freight train and engine, a pair of wheels with a heavy concentrated load should be employed to proportion our bridges.

Passenger engines with 40 000 lbs. on each of two axles are in use, and some designed with 45 000 lbs. on each axle.

Instead of introducing a second diagram to cover this case, it has been the writer's practice to require that two concentrated loads, at a fixed distance apart, shall also be considered.

For this, he requires, for the lighter specifications, that two 40 000-lb. loads, at a distance of 7 ft., must also be provided for, and for the heavier specification, two 50 000-lb. loads.

The writer would, therefore, offer as a basis for discussion the following specification for varying train loads:

1st.	Train load....	E. 25	with	two	40 000	lbs. at	7 ft. distance.
2d.	" " ..	E. 30 / E. 30	"		45 000	"	7 "
3d.	" " ..	E. 35 / E. 35	"		50 000	"	7 "
4th.	" "	E. 40					

TABLE No. 1.

Heavy Car Weights and Loadings.

Kind of Car.	No. of axles.	Length over all.	Weight of car.	Capacity of car.	Total weight loaded.	Load per foot of track.	Load per axle.
P. R. R. hopper gondola	4	29 2	23 200	60 000	83 200	2 853	20 800
E. T. V. & Ga. "	4	30 6	25 000	50 000	75 000	2 439	18 750
Ches. & Ohio "	4	30 9	24 000	60 000	84 000	2 732	21 000
Phil. & Read. "	4	35 4	22 800	60 000	82 800	2 343	21 200
Phil. & Read. coal car	4	23 6	18 480	56 000	74 480	3 170	18 620
L. S. & M. S. gondola car	4	35 6	27 150	60 000	87 150	2 455	21 790
L. S. & M. S ore car	6	34 3	36 500	100 000	136 500	3 985	22 750
Duluth & Iron Range ore car	4	24 6	24 400	53 760	78 160	3 190	19 540
L. & N. box car	4	36 8	28 000	60 000	88 000	2 400	22 000
S. P. box car	4	30 9	23 500	50 000	73 500	2 390	18 375
S. P. "	4	38 9	27 000	50 000	77 000	2 095	19 250
N. Y., L. E. & W. box car	4	36 5	31 900	60 000	91 900	2 524	22 975
P. R. R. box car	4	37 6	30 000	60 000	90 000	2 400	22 500
C., C. C. & St. L. box car	4	38 4	33 500	60 000	93 500	2 460	23 375
Street's stock car	4	40 4	29 800	60 000	89 800	2 226	22 450
Wicke's refrigerator car	4	35 6	42 300	50 000	92 300	2 600	23 075
P. R. R. refrigerator car	4	37 6	41 400	60 000	101 400	2 700	25 350
A., T. & St. Fé refrigerator car	4	37 6	41 500	40 000	81 500	2 173	20 375
P. R. R. gun truck, old	8	33	36 100	80 000	116 100	3 518	14 512
" " cable car	8	40 2	51 800	100 000	151 800	3 779	18 975
" " Krupp 120-ton gun	16	90 9	175 000	270 000	445 000	4 903	27 812
" " " 62-ton gun	12	70 9	113 300	140 000	253 300	3 580	21 108
" " single gun car	6	35 3	45 000	100 000	145 000	4 113	24 166
Baldwin's locomotive car	4	33	24 000	70 000	94 000	2 849	21 750

TABLE No. 2.

Classification of Freight Handled by Railroads.

The total freight handled during 12 months, amounting to 152 658 240 tons, upon the following railroads, viz.: New York Central and Hudson River R. R.; Lake Shore R. R.; Pennsylvania R. R.; United R. R.'s of New Jersey; Philadelphia and Erie R. R.; New York, Lake Erie and Western R. R.; Atchison, Topeka and Santa Fé R. R.; St. Louis and

San Francisco R. R.; Union Pacific R. R., and 8 400 miles of railroads in Iowa, is classified as follows :

Coal and coke	66 289 094 tons	44% of total
Grain	13 135 104 "	9% "
Lumber	11 588 910 "	7% "
Merchandise	20 427 453 "	13% "
Farm products	9 491 496 "	6% + "
Animal "	8 193 952 "	6% — "
Petroleum	1 868 190 "	1% "
Ores	5 438 494 "	3.5% "
Pig and bar metal	4 905 372 "	3% "
Mach'y and castings...	3 385 395 "	2% "
Stone, etc	7 934 780 "	5.5% "
Totals	152 658 240	100% "

TABLE No. 3.

Weight of Various Materials, Measured upon the Cubic Space Occupied by Them when Stowed in the Usual Packages.

Materials.	How packed.	Weight.
		lbs.
Coal, bituminous	In bulk	45 to 50
" anthracite	"	49 to 56
Coke	"	26 to 35
Lumber	"	30 to 45
Wheat, peas and beans	"	48
Corn and rye	"	45
Barley	"	38
Oats	"	26
Clover, flax and cotton seed	"	40
Hay	Bales	14
"	Compressed	24
Straw	"	19
Hops	Bales	11
Hemp	"	20
Wool	"	15 to 29
Tow	"	29
Excelsior	"	19
Cotton	"	12
"	Compressed	25 to 40
Leather	Bales	16
Raw hides	"	13
"	Compressed	23
Tobacco, leaf	Hogsheads or boxes	20
" plug	Boxes	56
Flour	Barrels	33
Pork and beef	"	40
Lard and tallow	"	40
Bacon	Bulk	38
Whiskey	Barrels	36
Sugar	"	42 to 45
Salt	"	48
Coffee	Bags	34
Cheese	Bulk	30
Oil cake	"	60
Oils	Barrels	35
Pitch and resin	"	48
Lime	"	50
Cement, American	"	56
" English	"	72
Plaster	"	53
Soda ash	"	62
Glass	Boxes	60
Crockery	Crates	40
Rope	Coils	42
Woolen goods	Bales	13 to 22
Cotton goods	"	16 to 37
Rags	"	15 to 23
Paper, newspaper	Rolls	38
" writing paper	Bundles	64
" wrapping paper	"	97
Nails	Kegs	63 to 74
Spikes	"	58 to 67
Rivets	"	91
Tin	Boxes	278
Railroad iron, 40-lb rails	In tiers	156
" " 100-lb. "	"	145
Pipe, cast iron, 6 ins. diameter	"	81
" " 12 "	"	62
" " 36 "	"	40
" " 48 "	"	35
" Wrought iron, 1 in. diameter	"	100
" " 3 ins. "	"	53
" " 6 "	"	40
" " 12 "	"	29
Bridge work, top chords	"	50 to 90
" floor beams	"	45 to 75
" pedestal castings	"	50 to 100
" eye-bars	"	325

TABLE No. 4.

Weight of Machinery per Square Foot of Floor Space.

Name of machine.			Weight per square foot floor.
Westinghouse engines,	4	H. P.....	121 lbs.
" "	25	"	178 "
" "	50	"	300 "
" "	100	"	303 "
Lidgerwood's H. engines,	6	"	241 "
" "	20	"	310 "
" "	50	"	361 "
" Up. Tub. boilers	5	"	202 "
" " "	15	"	203 "
" " "	35	"	259 "
" " "	60	"	280 "
" Sta. Loco. "	15	"	153 "
" " "	30	"	166 "
" " "	100	"	179 "
" Hor. Tub. "	20	"	103 "
" " "	100	"	159 "
Clerk gas engines,	5	"	117 "
" "	30	"	120 "
Gardiner's 3 Cyl. engines,	10	"	139 "
" "	60	"	214 "
New Acme Ver. engines,	5	"	156 "
" "	10	"	255 "
" "	20	"	287 "
N. Y. Safety St. Power V. Eng.,	5	"	160 "
" " "	10	"	190 "
" " Ver. boilers,	5	"	216 "
" " "	20	"	222 "
Holly Mfg. Co., heavy press. pumps	No. 1	.....	106 "
" " "	No. 13	.....	176 "
New Pulsometer pump	No. 5	.....	143 "
" "	No. 10	.....	308 "
Terracute Mach. Co. presses		.....	from 100 to 340 "
Bending, Forming and Shearing Mach.		.....	235 "
Nut tappers		.....	84 "
Emery machine		.....	86 "
Punching presses, heavy		.....	340 "

DISCUSSION.

GEORGE H. BLAKELEY, Jun. Am. Soc. C. E.—The discussion of this paper from the standpoint of the manufacturer may not be amiss. The almost endless variety in the existing specifications of live loads for railway bridges is a manifest absurdity of engineering practice that, for a long time, has been a subject of adverse criticism among the bridge builders and the engineers of the bridge building companies, as they have to bear the bulk of the burden which it imposes, and I can assure you it is no small burden.

It has for a long time been apparent to the bridge builders that it would be a comparatively easy task, were the engineers interested so inclined, to frame a standard system of typical loadings for railway bridges that would cover the ground as completely and thoroughly as the existing mass of specifications, and relieve them of a great quantity of entirely unnecessary and laborious calculations. In this matter the builder is at the mercy of the railroad companies. It is not for him to dictate. He must submit to the exactions of the purchaser. It is gratifying then, from our standpoint, to observe this tendency among the bridge engineers to simplify this question of the live loads for railway bridges, and bring order out of the existing chaos of almost infinite variety of specifications differing only by infinitesimal variations.

The paper on "Some Disputed Points in Railway Bridge Designing," by J. A. L. Waddell, M. Am. Soc. C. E., *Transactions*, Vol. XXVI, and which was discussed at great length by the members of this Society, was a long step in the right direction. That paper, however, covered too much ground and laid too great stress on the method of calculating the live load strains, and proposed a radical amendment in existing practice. The consensus of the opinion of those most competent to judge, and with evident correctness, condemns any departure from the employment of anything but typical engine loads in the design of railway bridges, not because of any superior ultimate accuracy that may be gained by their use, but by reason of their furnishing a readily understood standard that can be easily grasped, almost at once, by every railroad manager, being more or less a pictorial representation of actual loads, the comprehension of which is not confined to expert bridge engineers. It would be a serious mistake to abandon the use of typical loads. Having settled upon the retention of the typical engine load, the method of calculating the stresses is a question of minor importance, a secondary consideration, that can be left to the preference of the designer.

The paper presented seems to point the way to a simpler solution of this whole question of the proper live load for railway bridges than

any heretofore proposed. It places the whole question upon a more rational basis than any method yet proposed, and I am glad—heartily glad—that it has the sanction of such an eminent authority as Mr. Cooper. The bridge builders will await, with a great deal of interest, the opinions of the members of this Society as to its ultimate adoption.

In regard to the typical loading to be selected, doubtless we all have our own opinions ; but it does not seem that it will be very difficult to agree on one that will satisfy all, and meet all the requirements demanded ; and one for which every railroad bridge should, at least, be built, or made as much stronger as the requirements of the traffic demand, or the financial condition of the railroad permits. Mr. Cooper's proposed method of increase by percentages has many admirable points of merit which will commend it to every engineer familiar with the subject.

Mr. Cooper presents his proposed typical loading to this Society for general discussion. It is with this understanding that the following suggestions and information, gathered from locomotive builders, in reference to the general distribution of weight in the consolidation type of locomotive is presented for consideration.

The weight on the leading truck seems to be somewhat lighter than usual. The consolidation engine has, in general, about 86% of its total weight on the drivers when two of the driving axles are located under the firebox, and about 89% when the rear driving axle only is under the firebox. That is, the weight on the truck is 14% and 11%, respectively, of the total weight of the engine in the two cases cited. In the proposed loading the truck is only 9% of the total weight of the engine.

As a matter of convenience in calculation, the distance from the leading truck to the first driving axle might be made 8 ft. instead of 7½ ft., and thereby preserve integral wheel spacing throughout.

The driving wheel base of many of the existing consolidation locomotives of the lighter types is about 14 ft., but the mechanical engineer of one of the large locomotive companies assures me that the tendency is to the use of driving wheels 56 ins. in diameter, thus making 15 ft. the minimum rigid wheel base. In this light the use of 5-ft. centers of driving wheel axles seems well chosen as coinciding with the indications of present practice.

I believe that the proposed weight of tender should be increased somewhat to agree with actual conditions. Tenders are constructed at present much heavier, in proportion to the weight of engine, than formerly, and the tendency is to still heavier tenders. The New York, Lake Erie and Western Railroad is at present constructing a number of tenders weighing about 90 000 lbs., loaded, or about 22 500 lbs. on an axle. These tenders will be attached to their regular 55-ton engine, which has 28 000 lbs. on a driving axle. These phenomenally

heavy tenders are designed for use on a portion of the road where the distance between water stations will be longer than usual; still I believe that the weight of tender proposed in this paper should, at least, be increased $6\frac{2}{3}\%$, or so that the tender for engine 25, adopting the nomenclature of Mr. Cooper, would weigh 64 000 lbs., instead of 60 000 lbs. This would be the minimum increase, and I believe that an increase of 10% would not be out of the way. The following table gives the actual weights of several locomotives and the corresponding weights of accompanying tenders:

ACTUAL WEIGHTS OF LOCOMOTIVES.

Railroad.	Type of engine.	Weight on drivers.	Weight on truck.	Total weight of engine.	Weight of tender.
D., L. & W. R. R.	8-wheel Passenger	72 550	29 600	102 150	64 940
" "	" "	76 000	24 000	100 000	80 000
B. & O. R. R.	10-wheel	101 500	26 000	127 500	76 000
D., L. & W. R. R.	"	120 000	34 550	154 550	82 700
N. Y., L. E. & W. R. R.	"	111 750	26 350	138 100	79 600
" "	"	96 800	29 000	125 800	81 000
D., L. & W. R. R.	Mogul	97 770	16 200	113 970	75 590
" "	"	105 000	13 000	118 000	69 750
N. Y., L. E. & W. R R.	Consolidation	115 800	15 800	131 600	79 600
" "	"	110 700	19 500	130 200	79 600
C. & O. R. R.	"	110 800	16 500	127 300	72 000
Rogers Loco. Co.	"	108 000	12 000	120 000	68 000
W. N. Y. & P. R. R.	"	113 840	13 120	126 960	69 600

Some objection may be urged against the proposal of making the uniform train load a direct function of the weight of the engine, on the ground that there are exceptional cases of small or branch roads, hauling special freight heavier in proportion to the weight of the engine employed than is usual on the large or trunk lines. We must not lose sight of the fact that two consolidation locomotives coupled together, as the specification directs, cover a span of 100 ft., and the uniform train load is not considered in the calculation of the maximum stresses, unless the train load is heavier than the locomotives, a condition of affairs that is seldom or never realized in actual practice. In fact, the effect of the train load on spans of 125 ft., or even on spans of 150 ft., is inconsiderable. In this connection it is well to bear in mind that the great majority of railroad bridges are under 150 ft. span.

The provision for the effect of the heavy concentrations of the eight-wheel passenger locomotive, simply by the specification of two loads spaced a fixed distance apart, is to be commended from the calculator's standpoint, and is an instance of the elegance of the results that can be obtained by the use of the simplest methods. In marked

contrast to this is the practice of some bridge buyers, not content with the specifications of an anomalous loading, demand that their bridge shall be calculated for one or more additional loadings. We should bear in mind that complexity is not always an index of profundity, especially where simplicity serves the same end.

I believe that no railroad bridge, even for small or branch roads, should be constructed for a less load than engine 27 of Mr. Cooper's nomenclature. For some years I have recommended substantially this engine where the matter has been left to my judgment. I wish to urge Mr. Cooper, when he comes to the revision of this paper, to adopt an engine not less than engine 27, as a minimum standard typical loading for railway bridges.

This leads me to a subject that has not received the consideration that it should from the managers of the railways, and possibly many railroad engineers as well. It seems to be a general impression that the cost of a bridge is about or nearly in proportion to its capacity. Such is far from the truth. When the question is considered in detail this is evident. In building a bridge of any span, whether designed for one engine or for another, say 20% heavier, there are the same number of members to handle, practically the same number of holes to punch, ream or drill, and about the same number of rivets to drive in either case. There is practically the same amount of shop work on the lighter structure as there is on the heavier. The erection of the bridge in the field will cost about the same for both cases, as the most rational way of estimating the cost of erection is based on price per linear foot for similar structures. The only item of great importance, in increasing the cost of the heavier bridge, is the first cost of the additional material, and the freight will be increased, though not always, shipments sometimes being made by the car load lot, and the increase of weight for any moderate span would seldom demand an extra car. In connection with this subject, I have gone over the cost of several spans of various lengths recently constructed, in order to determine what the additional cost would have been had they been designed for heavier loads. I find that for plate-girder bridges the increase in first cost for each per cent. increase in capacity is $\frac{1}{4}$ of 1%, and for pin-connected bridges the increase is $\frac{1}{3}$ of 1% for each per cent. increase in capacity. That is, a bridge designed for engine 30 (Mr. Cooper's nomenclature) would only cost from 5% to $6\frac{2}{3}$% more than if designed for engine 25, which is 20% lighter. This is an important fact, and it should be emphasized. For an additional outlay of 5% for plate-girder bridges, or $6\frac{2}{3}$% for pin-connected bridges, an increase of 20% in the capacity of the bridge can be provided for. When this is more generally known and appreciated by our railway engineers and managers, we shall do much to inaugurate an era of stronger bridges on the smaller roads.

We are in a transition period from the use of iron to the general use of steel, and we are somewhat in danger of being misled by the results obtained from our specimens in the testing machines as indicative of the strength of the completed structure. We should be liberal in the loads for which we design our bridges, and, on the other hand, we should be conservative in the stresses that are allowed.

CHARLES S. CHURCHILL, M. Am. Soc. C. E. (by letter).—I heartily agree with the proposition which Mr. Cooper states as follows:

"Throwing aside, therefore, all references to relative merits of actual strains, diagrams, tables or other methods of rapidly approximating to the truth, let us take up and simplify the train loads by reducing them to their simplest forms and least number to cover the necessities of various classes of railroads."

I trust Mr. Cooper will not close this subject till he has reduced the train loads to absolutely their simplest forms. Although his proposed scheme of train loads shows a great increase in uniformity and system over his old specifications, he has not simplified it as far as is possible, as I will proceed to prove.

But, first, as to the weight of freight cars and freight trains of to-day:

The average load per foot of track of one of our regular coal trains is 3 200 lbs.; and we at times have several coal cars coupled together, producing a load of 3 350 lbs. per foot of track, the weight on one axle of a car being 24 500 lbs. Coal cars producing a load of 3 200 lbs. per foot are often transferred from this road to other connecting lines; and there are but very few railroads to-day that do not at times have freight trains averaging for several car lengths 3 200 lbs. per foot.

In view of this fact, I think it is unwise to specify for bridges a following train load of less than 3 500 lbs. per foot. On the other hand, I think 4 000 lbs. per foot is an ample specification loading to meet probable future increase in car loads.

Again, as to engine weights and their train loads, the heaviest engines do not always or even generally haul the heaviest cars. Their special use is to haul longer trains, or to overcome heavy grades on a given division of road; and, *vice versa*, the lightest engines do not haul the lightest cars; therefore, the train load per foot following any engine cannot be said to vary with the weight of the engine. In other words, whatever may be the weights of any engines used by any railroad of standard gauge, the probability is that they will at times haul several consecutive cars, producing a load of from 3 200 lbs. to 4 000 lbs. per foot of track.

I think, therefore, that the following train load is limited to the narrow margin of 3 200 lbs. and 4 000 lbs. per foot.

As to engines less need be said, excepting that a freight engine having four drivers of 28 000 lbs. seems to be about the safe minimum

for a specification, and every road must determine as near as possible what is to be the maximum, both for the entire line and for special divisions.

But now let us examine Mr. Cooper's proposed scheme of loading, taking the heaviest one, "E.-40," as typical of all. He shows two engines coupled, each having drivers of 40 000 lbs. and provided with tenders having axle loads of 24 000 lbs., and then a following train load of 4 000 lbs. per foot. The axle weight of this engine tender is no greater than that on one of our coal cars above referred to, and the load per foot produced by the tender is no greater than the following load of 4 000 lbs.; therefore, why should we not remove the engine tender from our consideration entirely? I made this point and described the practice of the Norfolk and Western Railroad in a previous discussion (see *Transactions*, American Society of Civil Engineers, Vol. XXVI, page 150), and quote the following two paragraphs:

"The advisability of removing the tender wheels of the typical loading was made evident from the fact that the weights on tender wheels are no greater than the weights on wheels of loaded freight cars; and that the increased train load per foot, which it is now found necessary to adopt generally, more than covers any concentration to be found under our tenders. This plan of forming diagrams like the above originated with my principal assistant engineer, Mr. Morgan E. Yeatman. The requirements of the problem at hand speedily showed to us its uses, and brought about the above results. There is no reason why, by the use of such diagrams, a simple type of loading, like that of our specifications, cannot be adopted by every railroad company to suit the requirements of their traffic.

"I trust this discussion may be continued until it results in the entire abandonment of a multiple form of engines for use in bridge specifications; and I hope that the above statement will indicate the uselessness even of the six types of locomotives proposed by Mr. Waddell for this purpose. I would recommend rather the use of a uniform load, with concentrations at not more than four points, which can be so selected as to cover the present and future requirements of the traffic of any railroad. The loading having been adopted, it is a simple matter to prepare diagrams to expedite the practical work of calculation of bridge strains."

I present herewith the Norfolk and Western Railroad Company's standard drawings, showing equivalent uniform loads per foot which will produce on any given span the same maximum bending moment and shearing force as the Norfolk and Western standard specifications of 1891, and several of the heavier types of engines in use at this date on the Norfolk and Western Railroad (Plates XX and XXI).

These diagrams show the engines which the specifications of January, 1891, were prepared to cover in advance of their construction, Class "G" being the heaviest in use at the date of the specification.

It will be noted that these diagrams show not only the effects of the various engines, but the present margin of safety in the specifica-

·Norfolk·&·Western·R·R·

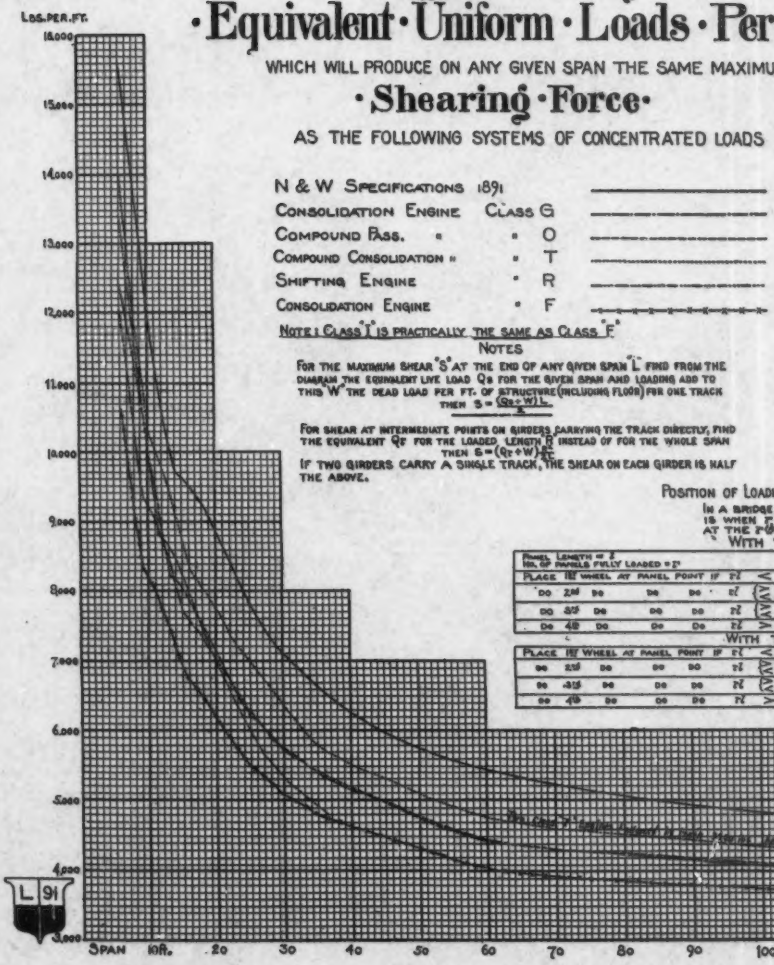
·Diagram·showing·

·Equivalent·Uniform·Loads·Per·Ft·

WHICH WILL PRODUCE ON ANY GIVEN SPAN THE SAME MAXIMUM

·Shearing·Force·

AS THE FOLLOWING SYSTEMS OF CONCENTRATED LOADS



N & W Specifications 1891		
Consolidation Engine	Class	G
Compound Pass.	"	O
Compound Consolidation	"	T
Shifting Engine	"	R
Consolidation Engine	"	F

Note: Class "T" is practically the same as Class "F"

Notes

For the maximum shear "S" at the end of any given span "L" find from the diagram the equivalent live load Q for the given span and loading add to this "W" the dead load per ft. of structure (including floor) for one track then $S = \frac{(Q+W)L}{2}$

For shear at intermediate points on girders carrying the track directly, find the equivalent Q for the loaded length "L" instead of for the whole span then $S = \frac{(Q+W)L}{2}$

If two girders carry a single track, the shear on each girder is half the above.

Position of Loading to give Max

In a bridge of 2' panels t
is when 7 panels are fu
at the 2nd panel point.

With the loading

Panel Length = 2' No. of panels fully loaded = 2'	No. of panels
Place 1st wheel at panel point if 2'	2
Do 2.86 Do Do Do 0'	2-0
Do .572 Do Do Do 2'	2-4
Do 4.0 Do Do Do 2'	2-4

With the loading

Place 1st wheel at panel point if 2'	
Do 2.86 Do Do Do 2'	
Do .572 Do Do Do 2'	
Do 4.0 Do Do Do 2'	

Western·R·R·

n · showing ·

form · Loads · Per·Ft ·

ANY GIVEN SPAN THE SAME MAXIMUM

ing · Force ·

SYSTEMS OF CONCENTRATED LOADS

1891 ———————

CLASS G ———————

" O ———————

" T ———————

" R ———————

" F ———————

THE SAME AS CLASS "F"

NOTES

THE END OF ANY GIVEN SPAN "L" FIND FROM THE
Q's FOR THE GIVEN SPAN AND LOADING ADD TO
OF STRUCTURE (INCLUDING FLOOR) FOR ONE TRACK
S = (Qs + W) L

TS ON GIRDERS CARRYING THE TRACK DIRECTLY, FIND
ADED, LENGTH L INSTEAD OF FOR THE WHOLE SPAN
S = (Qs + W) L
GLE TRACK, THE SHEAR ON EACH GIRDER IS HALF

POSITION OF LOADING TO GIVE MAXIMUM SHEAR
IN A BRIDGE OF 25' PANELS THE MAXIMUM SH
IS WHEN 7 PANELS ARE FULLY LOADED AN
AT THE 8th PANEL POINT.

WITH THE LOADING OF N&W S

PANEL LENGTH = d NO. OF PANELS FULLY LOADED = 7							No. OF PANELS IN "						
PLACE 1ST WHEEL AT PANEL POINT					IF		3	4	5	6	7	8	9
00	2nd	00	00	00	2'	AVA	9'-4"	14'-0"	52'-2	58'-8	65'-2	69'-6	84'-
							34'-8	21'-6	68'-0	86'-0	104'-8	122'-2	144'-
00	3rd	00	00	00	2'	AVA	34'-8	21'-6	68'-0	86'-0	104'-8	122'-2	144'-
							34'-8	21'-6	68'-0	86'-0	104'-8	122'-2	144'-
00	4th	00	00	00	2'	A	34'-8	"	"	"	"	"	"

WITH THE LOADING OF CLASS "T"

PLACE 1ST WHEEL AT PANEL POINT					IF		6-0	11'-6"	11'-7"	141-2"	18'-2"	18'-8"	21'-
00	2nd	00	00	00	2'	AVA	6-0	11'-6"	11'-7"	141-2"	18'-2"	18'-8"	21'-
							0-0	25'-9	46'-8	57'-8	64'-2	71'-0	84'-
00	3rd	00	00	00	2'	AVA	0-0	25'-9	46'-8	57'-8	64'-2	71'-0	84'-
							0-0	"	"	"	"	"	"
00	4th	00	00	00	2'	A	0-0	"	"	"	"	"	"

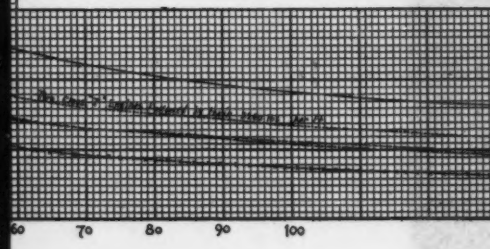
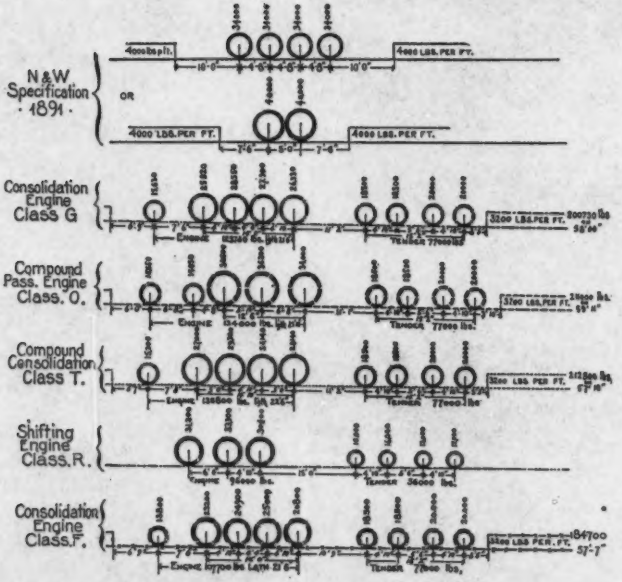


PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 692.
CHURCHILL ON TRAIN LOADINGS FOR BRIDGES.



MUM SHEAR IN A GIVEN INTERMEDIATE PANEL.
E MAXIMUM SHEAR IN THE $(n+1)^{th}$ PANEL
LY LOADED AND ONE OF THE LOADS STANDS

OF N&W SPECIFICATION 1891

												Shear from live load only for one track in lbs.
8	7	6	5	4	3	2	1	0				
0	0	0	0	0	0	0	0	0				

CLASS "T" ENGINES

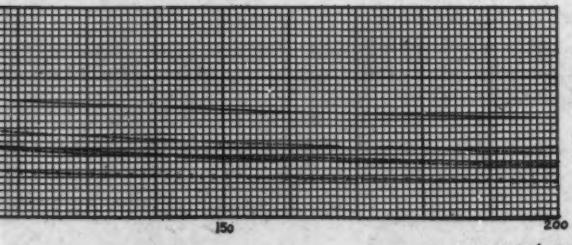
8	7	6	5	4	3	2	1	0				
0	0	0	0	0	0	0	0	0				

Approved:

Vice Pres. & Gen. Manager

Correct:

Engr Maintenance of Way.



Office of Engr Maint. of Way
Roanoke, Va. Sept. 1893

tion over Class "T" engines. It will be further noted that our very simple form of specification loading produces a load line of practically the same general form as the actual engines and trains. These diagrams alone prove that just such a simple specification loading may be found to cover any engine (actual or typical) desired.

To demonstrate this I have had prepared a similar diagram (Plate XXII) to cover Mr. Cooper's loading, "E.–40." This is shown thus: —— —— ——(3). On this drawing is also shown, for comparison, the diagram for Norfolk and Western Class "T" engine, thus :..... ..... (2); and Norfolk and Western specification loading, thus: ———————— (1); and, as a matter of interest, diagram of Mr. Cooper's loading, "E.–35," is also shown, thus:——*——*——*——*—— (4). This last, it will be noted, very nearly corresponds with the diagram of the Norfolk and Western specifications, but does not follow the diagrams of the actual engines so uniformly.

Now, to substantiate my first statement, there is shown on this same drawing a diagram indicated thus, —— – —— – —— – —— (5), which is almost identical with the diagram of Mr. Cooper's loading, "E.–40" [—— —— —— —— (3)], and the corresponding shear diagrams will also be found to be in close accordance. The loading, however, from which the former was produced is far simpler than Mr. Cooper's "E.–40." Instead of two engines and tenders followed by a uniform load, involving much needless calculation with every bridge, it is simply four loads of 40 000 lbs., 5 ft. apart, replacing a length of 27 ft. of the uniform load of 4 000 lbs. per foot; or, for short spans, an alternative of two loads of 50 000 lbs. 7 ft. apart.

Now, if Mr. Cooper will take this one further step in advance, and get rid of the useless typical engines and tenders, except for a means of reference, using simply their equivalent, which may always be found as above, by not more than four loads uniformly spaced, and replacing some lengths to be determined of this uniform load, he will answer his own proposition and receive the commendation of many railroad and bridge engineers.

I have not in the above discussed the loading "E.–40" in general. It may, however, be of interest to note that the diagram of this load, or my proposed equivalent, gives a slightly greater percentage of excess over trains in actual use on long spans than on short spans. A better typical loading to replace "E.–40" would be four loads of 40 000 lbs. each, spaced 5 ft. apart and replacing 30 ft. of the uniform load of 4 000 lbs., shown by a line thus: —— - - —— - - —— - - —— (6).

It might be well to add that the reason for adopting concentrations at four points instead of a uniform load with a single concentration, as has often been suggested, is that this latter, though it may be so chosen as to represent the equivalent of the actual loading for bend-

ing or for shear separately, will never represent the two correctly at the same time. If chosen as a correct equivalent for bending moments, it will invariably give too low results for shear.

I close by again referring to the last clause quoted above from my previous discussion on this general subject.

JOHN A. FULTON, M. Am. Soc. C. E. (by letter).—The series of engine loads outlined by Mr. Cooper, with his percentage method of variation, ought to afford an excellent assortment to select from; but when we come to consider train loads per linear foot, there appears to be no good reason for assuming that they should bear any specific ratio to the engine loads.

The difference between a freight train on a great through line having light grades, and a freight train on a mountain road having heavy grades, is in the number of cars per train, and not in the loads on individual cars. It would therefore seem that the typical freight train load per linear foot which would be proper for one road should be about the same for any other road.

While any considerable increase in the dimensions of the present box car is not probable, there seems to be no difficulty in the way of increasing the height and carrying capacity of a coal car, although it is now among the heaviest cars in use.

It would be making an important step in the right direction if we could adopt an unvarying freight car load for all standard gauge railroads, and fix its amount at somewhere about 4 000 lbs. per linear foot.

This would be an ideal train load. It would include the effect of many a flat car wheel and would probably be ample for all purposes; but it would seem that anything under 3 000 lbs. per linear foot would be making very little allowance for future possibilities or for present excesses.

It is to be remembered that the average bridge span is not a very long one, that three or four cars will more than cover it, and that in making up trains the yardmen are not supposed to spend much time distributing their heaviest cars through the train in such a way as to keep them separated from each other; and, in consequence, any long train is as likely as not to have its heavy loads bunched in one portion of the train and to produce as ill effects on the bridges as if the whole train were excessive in weight per linear foot. It would also be quite in the line of progress to abandon altogether the practice, now in vogue on many railroad systems, of using a certain engine load for bridges on main lines, and some other and lighter load for bridges on branch lines.

As it is true that a heavy passenger engine may often be sent over an unimportant branch line, so it is also true that a heavy freight engine may often be sent over that same branch line.

The motive power of a railroad is never in charge of the bridge

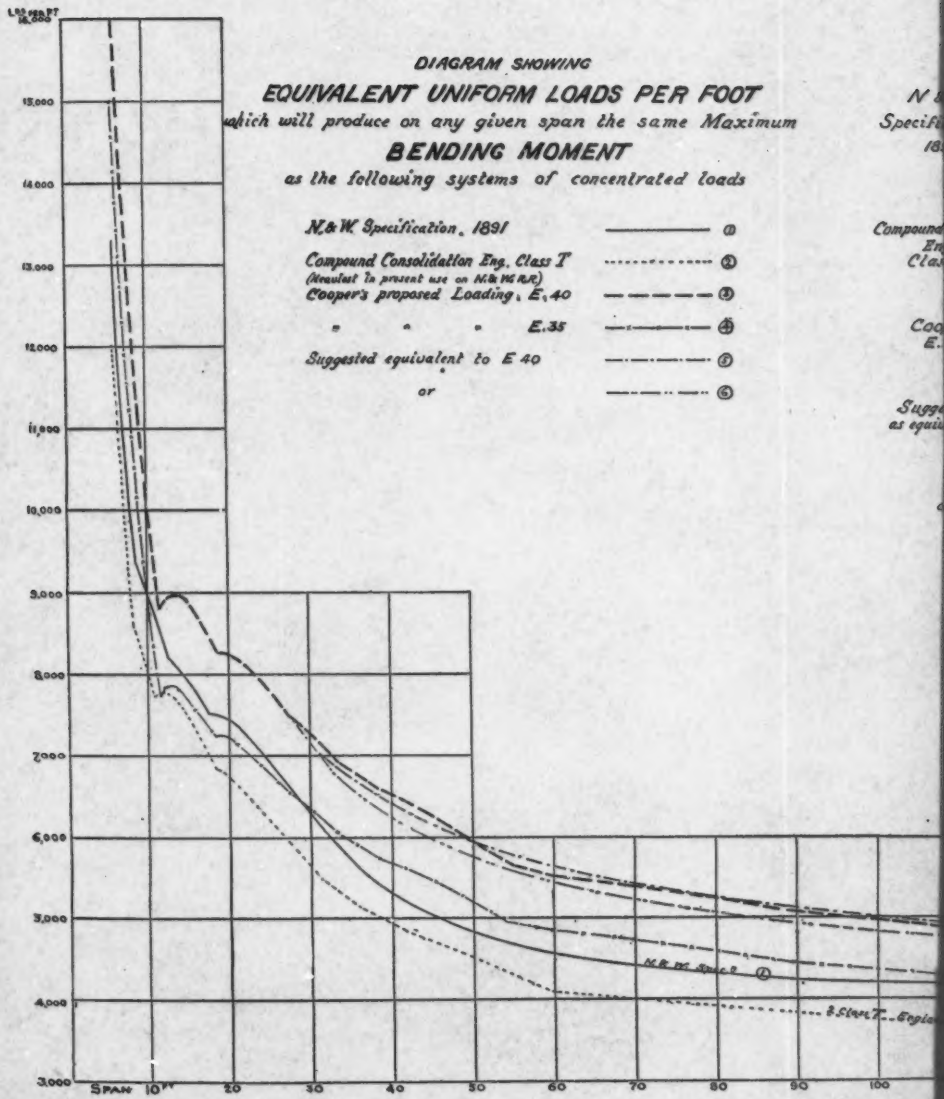


DIAGRAM SHOWING
EQUIVALENT UNIFORM LOADS PER FOOT
which will produce on any given span the same Maximum
BENDING MOMENT
as the following systems of concentrated loads
N.& W. Specification, 1891 ①
Compound Consolidation Eng. Class T ②
(Nearest to present use on N.& W. R.R.)
Cooper's proposed Loading, E.40 ③
" " " E.35 ④
Suggested equivalent to E 40 ⑤
or
Lbs per Ft
18,000
17,000
16,000
15,000
14,000
13,000
12,000
11,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
Span 10 Ft
20
30
40
50
60
70
80
90
100

M SHOWING

ORM LOADS PER FOOT

iven span the same Maximum

G MOMENT

ems of concentrated loads

N & W Specification 1891

91 ——— ①

Compound Consolidation Engine Class T

ng. Class T ..... ②
& W R R)

ing. E. 40 — — — — ③

E. 35 —·——·— ④

E 40 —·—·—·— ⑤

—·—··—·— ⑥

Cooper's E. 40 ③

Suggested as equivalent ②

or ⑥

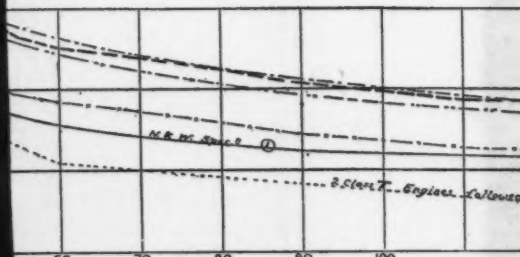
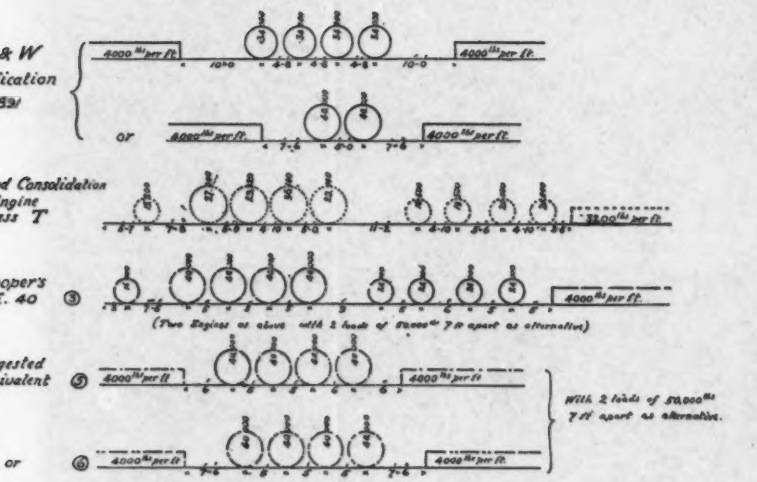


PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 692.
CHURCHILL ON TRAIN LOADINGS FOR BRIDGES.



department, and when the master mechanic has a call for an engine he sends out the one that seems to him the most available, regardless of its effects on the bridges.

EDWIN THACHER, M. Am. Soc. C. E. (by letter).—Of all plans which have been proposed for simplifying train loadings on railroad bridges, the writer considers the plan proposed in this paper by far the simplest and best, and he can see no reason why it cannot be consistently and profitably adopted by every railroad company in the United States. The wheel spacings and the distribution of load on the different wheels appear to be well chosen, so far as the writer can judge, except that he would consider a distance of 8 ft. between axles for the concentrated loads safe; this, however, is of little consequence, as these loads apply only to very short spans.

As it is desirable to know, without trial, which of the two loads will produce a maximum moment, the following formulas are given:

Let W = weight on one driver concentrated load,
" W = " " " freight engine,
" L = length of span in feet,
" M = bending moment in foot-pounds,—then

Concentrated load 1 driver on span $M = W\frac{L}{4}$ ..... (1)

" 2 " " $M = \frac{W}{2}\frac{(L-3.5)^2}{L}$ ..... (2)

Freight engine 3 " " $M = W\left(\frac{3L}{4} - 5\right)$ .... (3)

" " 4 " " $M = W\left[\frac{(L-2.5)^2}{L} - 5\right]$ (4)

Equating (1) and (2) $L = 11.95$ ft.
" (2) and (4) $L = 18.66$ " hence the following:

Concentrated loads $\begin{cases} L < 11.95 \text{ use (1)} \\ L > 11.95 \text{ " (2)} \end{cases}$
Freight engine $\begin{cases} L < 18.66 \text{ " (3)} \\ L > 18.66 \text{ " (4)} \end{cases}$

For load "E.-25" $W = 20\,000$, $W = 12\,500$
Equating (2) and (4) $L = 22.78$.
For load "E.-40" $W = 25\,000$, $W = 20\,000$,
Equating (1) and (3) $L = 11.43$.

As the concentrated loads do not apply to spans exceeding 22.78 ft. for the lighter engine, or to spans exceeding 11.43 ft. for the heavier engine, they are practically confined to the calculation of track stringers of moderate length, and cause no complication in bridge stresses.

WARD BALDWIN, M. Am. Soc. C. E. (by letter).—I have read Mr. Cooper's paper with great interest on account of the valuable statis-

tics it contains, and also because the same plan Mr. Cooper suggests for arranging a set of bridge loadings, viz., grading them so that the parts of each loading have a uniform ratio to the corresponding parts of any other loading of the set, was adopted by the writer in a set of loadings he sent, in January, 1893, to J. A. L. Waddell, M. Am. Soc. C. E., as a suggested modification of Mr. Waddell's proposed set of standard loadings. If a set of loadings, such as Mr. Waddell has proposed, meets all requirements, then the advantages of such a modification seem to be sufficient to warrant its adoption; and it might be of interest to hear from Mr. Waddell at this time why he rejected this suggestion. If present practice, however, may be accepted as a fair criterion of the general limits a set of standard loadings should cover, the conclusion seems inevitable that a set of loadings graded so that the uniform load bears a uniform ratio to the driver load will not include all the live loads that ought to be specified. With the purpose of ascertaining what conditions a set of standard loadings should fulfill, the writer secured copies of the bridge specifications in force in 1893 on 100 independent railway systems in this country, and made a comparative study of the loadings therein. A full list of these loadings and a proposed set of standards were given by the writer in a discussion of the paper by C. D. Purdon, M. Am. Soc. C. E., on "Comparison of Modern Engine Loadings, with Standard Specifications for Spans from 10 to 200 Feet," read at the Chicago Congress. A glance at this list of loadings will make it plain that in the specifications now in use the uniform load is not even approximately proportional to the engine loading.

For instance: Twenty-one specifications unite a 30 000-lb. driver load to a 3 000-lb. uniform load, and 14 unite the same driver load to a 4 000-lb. uniform load.

Again: The Pennsylvania Railroad unites a 30 000-lb. driver load to a 4 000-lb. uniform load; while the Atchison, Topeka and Santa Fé Railroad unites a 44 000-lb. driver load to a 3 200-lb. uniform load. Unless it is true that local conditions do not require the coupling of heavy engines to light trains, and *vice versa*, then the set of loadings proposed by Mr. Cooper would not appear to be comprehensive enough. Mr. Cooper's statement that "there is far greater probability of harmony on this subject than on many other disputed points in practice" encourages the writer to reiterate the hope that this question will be referred to a committee of the Society.

J. C. Bland, M. Am. Soc. C. E. (by letter).—In Mr. Cooper's paper, "Train Loadings for Railroad Bridges," the writer thinks there is a means of arriving at an agreement as to a series of standard loadings, especially so as the author has distinctly eliminated the already much-discussed question as to which of several methods is best to follow in obtaining stresses. The subject to which the author has confined him-

self is largely a question of fact and of familiarity with the several kinds of rolling stock in general use; and the matter for discussion is then on the possibility of representing by a base loading two consolidation engines and their appropriate car loads; whether the latter can be expressed by a constant ratio to the engine driver loads, and whether the tenders and truck axle loads can likewise be expressed by another constant ratio. Both of these questions involve some difficulty, for, with practically the same engines, one road demands greater tender capacity (coal and water) than another, due entirely to differences in physical conditions, and by no means connected with character of car loading. Some variations in the ratio of tender to driver axle loads are shown in Table No. 1 (page 196).

Another point which somewhat complicates the question is, whether, in a series of standard loadings, a constant wheel base spacing can be maintained for the engines as the driver loads increase in weight? An examination of data of actual engines shows a fairly constant rigid wheel base of engine, and also of tender, *per se* but an increase of from 20 to 30% in the space between rear driver and first tender axle as the engine becomes heavier; in short, the engine length increases with the weight. For some data regarding this see Table No. 1. In the opinion of the writer neither question is of serious importance, for, in the case of the tender capacity, a development of water and coal stations, at any time likely to take place on a given line, reduces the objection, though the tank capacity remains; and as regards the increased wheel base for the heavier engines, it only at the most, amounts to 2½ or 3 ft., or an increase of, say, 5% in total engine length.

The other question to settle is, having a given consolidation engine, what train load is it proper to consider as following it? Mr. Cooper considers the appropriate train one which weighs per linear foot one-tenth the weight on the driver axles. The writer has worked for some time on this subject, but from a somewhat different starting point, not the weight per foot of train, but the weight per car axle, desiring to express the latter by a constant ratio to load on driver axle. He, therefore, by so doing, had also to fix on a standard car length and axle spacing, and in consequence of selecting the hopper-bottom coal car as a type of spacing, gets somewhat heavier car loads per linear foot to accompany a given engine than Mr. Cooper, viz., about 11% greater. Mr. Cooper's remarks, on pages 179 and 180, together with his tables, leave little to be said on the question of car storage capacity. The 4 000-lb. train load, which he places as a maximum, gives on his assumed car length of 37 ft. a total of 148 000 lbs., that is, 37 000 lbs. on each of four axles. The writer having taken—corresponding to a 40 000-lb. driver load—a car axle load of five-sixths, that is, 33 333 lbs., gets, by reason of the assumed car length of 30 ft.,

TABLE No. 1.

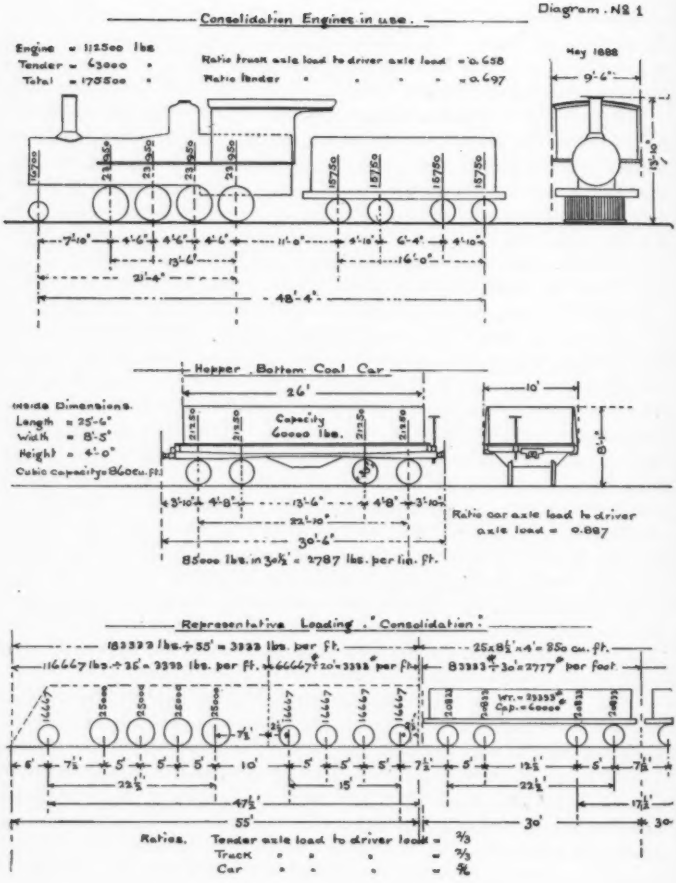
DATA ON CONSOLIDATION ENGINES.

NAME.	Date.	WEIGHT IN WORKING ORDER.						TENDER.		Cylinder.	WHEEL BASE.				
		On truck.	On drivers.	On engine.	On tender.	Total engine and tenders.	Ratio tender to drivers.	Water galls.	Coal, lbs.		Engine.		Between engine and tender.	Tender.	Total engine and tender.
											Rigid	Total.			
West Shore	1883	16 000	88 000	104 000	64 000	168 000	0.727	3 000	10 000	20" x 24"	14' 0"	21' 7"	10' 4"	15' 8"	47' 7"
Class. I, Penna. R. R.	1876	12 240	79 400	91 640	55 770	147 340	0.692	3 000	8 000	20" x 24"	13' 8"	21' 6"	10' 9"	15' 4"	47' 7"
Class I, Penna R R	1886	13 000	82 700	95 700	57 800	153 500	0.699	3 000	8 000	20" x 24"	13' 8"	21' 6"	10' 9"	15' 4"	47' 7"
Class. R, Penna. R. R.	1886	14 025	100 600	114 625	57 800	172 425	0.574	3 000	8 000	20" x 24"	13' 10"	21' 9"	11' 8"	15' 4"	48' 9"
Class. 112, D. & R. G. R R.	1888	16 700	95 800	112 500	68 000	180 500	0.710	.....	.....	20" x 24"	13' 6"	21' 4"	11' 0"	16' 0"	48' 4"
Class. 113, R. G. W. Rwy	1891	16 700	98 300	115 000	68 000	183 000	0.691	3 200	15 680	20" x 24"	13' 6"	21' 6"	11' 0"	15' 10"	48' 4"
Colorado Midland	1888	12 000	114 000	126 000	82 000	208 000	0.719	4 000	16 800	20" x 26"	14' 6"	22' 0"	10' 6"	15' 6"	48' 0"
Baldwin "Octopod"	1890	13 700	133 800	147 500	82 800	230 300	0.619	....	.....	.....	14' 0"	22' 3"	12' 1½"	14' 5"	48 9½"
Reading, Class. I, 75	1888	14 700	138 334	153 054	71 980	225 014	0.520	.....	.....	.....	14' 0"	22' 3"	13' 3½"	15' 8"	51' 2½"
Reading, Class. I, 55	1888	12 500	110 300	122 800	64 140	186 940	0.581	.....	.....	.....	13' 8"	21' 9"	12' 3½"	15' 7"	49' 7½"
Reading, Class. I, 53	1888	15 380	104 120	119 500	64 140	183 640	0.616	.....	.....	.....	13' 8"	21' 9"	12' 3½"	15' 7"	49' 7½"
Reading, Class. I, 50	1888	15 380	101 470	116 850	64 140	180 990	0.632	.....	.....	....	13' 8"	21' 9"	12' 3½"	15' 7"	49' 7½"
Baldwin, 10-34 E., 1 271 to 1 290	1891	16 000	96 000	112 000	74 000	186 000	0.772	.....	.....	.....	13' 6"	21' 4"	13' 4"	16' 0"	50' 8"
Western N. Y. & P. Compound No. 175	1891	13 050	116 550	129 600	.....	.....	.....	.....	.....	.....	14' 0"	21' 9"			
Western N. Y. & P. Compound No. 169	1891	16 220	110 650	126 870	.....	.....	.....	.....	.....	19" x 26"	14' 0"	21' 9"			
Ills. Central	1892	18 700	118 600	137 300	.....	.....	.....	3 850	.....	.....	16' 9"	24' 5"			

a deduced load per linear foot of 4 444 lbs., that is, an 11% greater load per linear foot from a 10% less axle load. The ordinary freight train is a medley of various sorts, kinds and lengths of cars from all over the country—box, stock, flat and coal cars mixed; except as regards the large house cars of western lines, there seems no limit to the interchange of car stock. The writer has seen Lehigh Valley coal cars in the middle of Utah. Excluding coal or ore carrying roads, on the average line, it is only at certain seasons of the year that one sees continuous trains of the heavier class of freight, viz., coal, ore and the like.

Interchange of engines, being comparatively rare, may be considered as inadmissible, though now and again, as Mr. Cooper points out, passenger engines travel over other lines than their own; and for this reason the writer considers, no matter what the freight traffic may be, that it is prudent to design the bridges for two such concentrated axle loads as has been Mr. Cooper's practice, especially as it only affects the short spans. Corresponding to the loading, Class E.-30, of Mr. Cooper, the writer has used two concentrated axle loads of 50 000 lbs. each, spaced 8 ft. apart, which recently he changed to 7½ ft. Though there is little opportunity for increase in the widths or heights of cars in the East, in the West, while the widths would remain fixed, there is room for increase in height, as the standard headroom above rail is greater. The Santa Fé requires 22 ft., the Denver and Rio Grande Railroad and Rio Grande Western Railway require 21 ft. The highest freight car which the writer has seen is one 13 ft. 6 ins. from top of rail to top of running board, to which, adding 6 ft. for a man, we get 19 ft. 6 ins., which leaves with above headrooms, at least 1½ ft. for increase; and as such a car has about 8¾ to 9 ft. clear inside height, such height can be increased at least 15%; and, therefore, the storage capacity by that percentage. Though this be the fact, the writer doubts its utility, for to load such a storage space with freight of the heavier class would, by the increase of axle loads, demand heavier rails than are customary in the West. The rule seems to be, the longer and larger the car, the lighter its axle load, for such cars are designed only for the lighter freight.

The writer for a long time having had the hope of being able to express the spacing of the axle loads of engines, tenders and cars in terms of the distance between the drivers, and also the loads on each axle in terms of the load on each driving axle, had collected and tabulated all the data obtainable regarding the rolling stock of several first-class railways, noting at the same time the characteristics concerning gradients, curvature and the nature of traffic. It was thought that all axle spacings might be found—with such a degree of approximation as seemed necessary or desirable—to be multiples of 2 or of 2½ ft., and also that the ratio of tender load to driver load would be close to two-thirds. The data tabulated comprised that for 8-wheel, 10-wheel and consolidation engines, a later page of the latter being given


Consolidation Engines in use.
Diagram. N° 1
Engine = 112500 lbs
Tender = 63000 "
Total = 175500 "
Ratio truck axle load to driver axle load = 0.658
Ratio tender " " " " = 0.497
Hey 1888
9'-6"
13'-10"
48'-4"
Hopper Bottom Coal Car
Inside Dimensions.
Length = 25'-6"
Width = 8'-5"
Height = 4'-0"
Cubic capacity = 860 cu. ft.
Capacity 60000 lbs.
26'
10'
8'-1"
Ratio car axle load to driver axle load = 0.887
85000 lbs. in 30½' = 2787 lbs. per lin. ft.
Representative Loading "Consolidation"
183333 lbs. ÷ 55' = 3333 lbs. per ft.
116667 lbs. ÷ 35' = 3333 lbs. per ft.
66667 ÷ 20' = 3333" per ft.
83333 ÷ 30' = 2777" per foot
25 x 8½' x 4' = 850 cu. ft.
Wt. = 83333 lbs.
Cap. = 60000
Ratios. Tender axle load to driver load = ⅔
Truck " " " " = ⅔
Car " " " " = ⅚

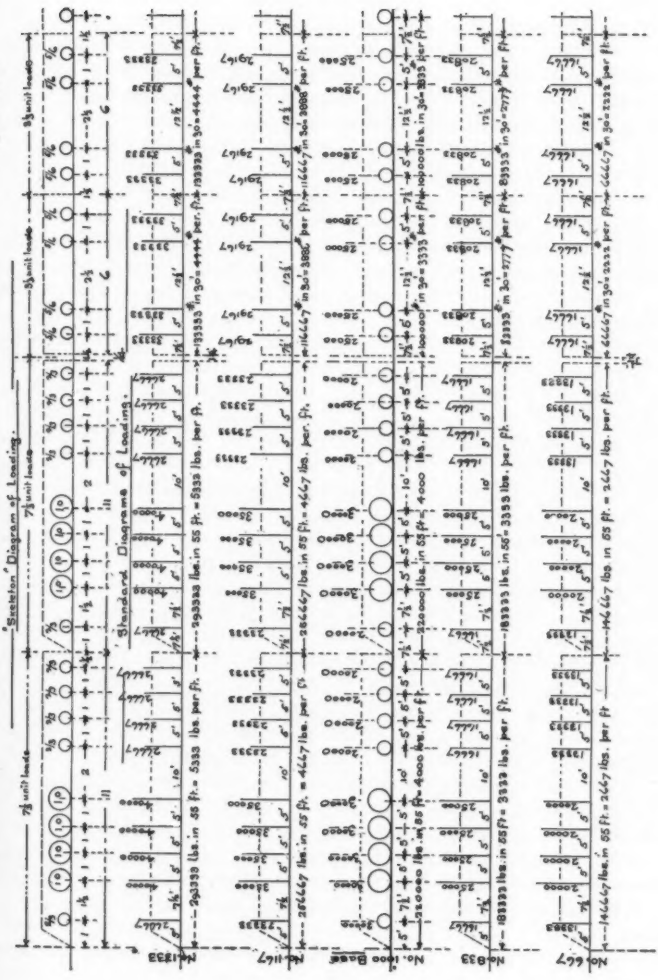


Diagram N° 2.
'Skeleton' Diagram of Loading.
Standard Diagrams of Loading.
No. 1333
No. 1167
No. 1000 Base
No. 833
No. 667

in Table No. 1 (page 196). From this table the writer concluded that a standard spacing for consolidation engines could be as shown on the "Representative Loading," foot of Diagram No. 1 (page 198), wherein it is seen that all distances are multiples of $2\frac{1}{2}$ ft.

Regarding the axle loading, it was concluded that, taking the driver weight as unity, the loads on the truck axle and on each tender axle could be represented by the fraction $\frac{3}{4}$. That this fraction was in general too large in case of the truck axle load was realized; but inasmuch as it was the writer's practice to disregard the truck load in determining the shears, though including it in computing the cumulative moments and the reactions, it was allowed to stand. Whence the weight of a consolidation engine was taken to be $4\frac{3}{4}$ units, the tender to be $2\frac{3}{4}$ units, and the total weight of engine and tender to be $7\frac{1}{2}$ units, the unit being the load on each driver axle.

As to the spacing, the engine wheel base was taken $4\frac{1}{2}$ units, the tender wheel base 3 units, the distance from engine to tender 2 units, the total wheel base of engine and tender $9\frac{1}{2}$ units, and total length of engine and tender to be 11 units, the unit spacing being the distance between the driving-wheel axles. A "Skeleton Load Diagram" being then formed (see Diagram No. 2, page 199), the writer computed the cumulative moments at each axle in terms of the driver axle load as unity and driver axle spacing as unity. From this table of "skeleton moments" the moments for any consolidation engine could readily be found by multiplying the tabular "skeleton" moment by the product of the driver axle load and driver axle spacing; or rather the "skeleton" stresses were determined in any instance, and then multiplied by the foregoing product for the stresses due to engine in question. For example: If 5 ft. be taken as the spacing of the driver axles, then the factor by which to multiply the "skeleton" moments is five times the driver axle load; or if, as some have maintained, we should adhere to $4\frac{1}{2}$ ft. as the driver distance, then the factor would become four and one-half times the driver axle load. In the first instance the actual spacing would be as shown on the five standard load diagrams of Diagram No. 2 (page 199); in the second instance the spacings would be 90%; *i. e.*, the wheel distances would reduce respectively to 4.5 ft., 6.75 ft., 3 at 4.5 ft., 9.0 ft., 3 at 4.5 ft., 2.25 ft. = 49.50 ft. total length. For a narrow-gauge engine, the driver distance could be taken 4 ft., whence the following spacing, 4.0 ft., 6.0 ft., 3 at 4.0 ft., 8.0 ft., 3 at 4.0 ft. 2.0 ft., = 44.0 ft. total length; that is, 80% of the spacing on the Standard Load Diagrams, which agrees closely with existing narrow-gauge engines.

An examination of the tabulated car stock data collected indicated considerable variation in loading and spacing of cars of the same capacity and type, which the writer concluded was largely due to the precedent and peculiarities ruling on each road; he therefore

concluded to await an opportunity to secure data of recent car stock where such restrictions did not obtain. Such an opportunity presented itself about three years ago when new stock was being purchased by a road with the characteristics of which the writer was familiar; and there was this further advantage, the motive power was bought about the same time and was designed by the Superintendent of Motive Power and Machinery, who was not only thoroughly versed in all modern practice, but was also particularly well informed as to the requirements of his own line. Table No. 2 gives the car

TABLE No. 2.

STANDARD GAUGE FREIGHT CARS, D. & R. G. R. R.

	Coal Cars.		Low flat.	Box car.	Stock car.
	Gondola.	Hopper-bottom.			
Outside dimensions—Length 0 to 0 B. N.	34′ 6″	30′ 6″	34′ 8½″	34′ 6″	34′ 6″
" " Width 0 to 0	9′ 8″	10′ 0″	9′ 2″	9′ 3″	9′ 3″
" " Height 0 to 0	7′ 7½″	8′ 1″	3′ 4″	11′ 6″	11′ 6″
Inside dimensions—Length	30′ 0″	25′ 6″	32′ 0″	31′ 5⅜″	31′ 5″
" " Width	8′ 2″	8′ 5″	8′ 10″	8′ 1⅜″	8′ 1″
" " Height	3′ 6″	4′ 0″	.....	6′ 1½″	6′ 1½″
" " Cubic contents	858	860	.....	1 568	1 556
" " Cross-section	28.6	33.7	.....	49 9	49.5
" " Floor area	245	215	283	256	254
Wheel base—Truck	4′ 8″	4′ 8″	4′ 8″	4′ 8″	4′ 8″
" Total	27′ 2″	22′ 10″	27′ 2″	27′ 2″	27′ 2″
Weights—Dead weight	24 000	25 000	21 000	26 000	26 000
" Nominal loading	60 000	60 000	60 000	60 000	60 000
" Total	84 000	85 000	81 000	86 000	86 000
Deductions—Dead weight per foot of length	696	820	605	754	754
" Nominal loading of length	1 739	1 967	1 729	1 739	1 739
" Total " "	2 435	2 787	2 334	2 493	2 493
" Dead weight on each axle	6 000	6 250	5 250	6 500	6 500
" Live " "	15 000	15 000	15 000	15 000	15 000
" Total load "	21 000	21 250	20 250	21 500	21 500
" Cubic capacity per foot of length	24.9	28.2	.....	45.5	45.1
" " " height	245	215	.....	256	254
" Nominal loading÷Cubic capacity	70	70	.....	38	39
" Dead weight÷Nominal loading	0.4	0.417	0.35	0.433	0.433
" " ÷Total loading	0.286	0.294	0.259	0.302	0.302
" Nominal loading÷Dead weight	2.5	2.4	2.86	2.31	2.31

stock referred to, the consolidation engine in use being shown at head of Diagram No. 1 (page 198).

The writer considered a representative trainload to be one composed of a series of hopper bottom coal cars, a diagram of one of which is shown underneath the consolidation engine, and from these two he formed a "Representative Load Diagram," two consolidation engines heading a train of hopper-bottom coal cars (see Diagram No. 1, page 198), in which the spacing throughout is a multiple of 2½ ft., the loading on truck and each tender axle two-thirds the load on driver axle, and

the load on each car axle five-sixths the load on each driver axle. This loading being representative of the actual traffic on the line, for proportioning the bridges, it was considered prudent to increase it 20%, whence the "typical load diagram" is shown on Diagram No. 2 (page 199), where it is called "No. 1 000 Base."

From this typical load diagram as a base (viz., two 220 000-lb. engines heading a train of coal cars, each being 100 000 lbs. in 30 ft.), a series was constructed, each one varying from the base load by one-sixth and one-third, above and below, diagrams of which are shown in Diagram No. 2, numbered above the base 1 167 and 1 333, and below the base by 883 and 667, the numbers being indicative of the ratios to the base load.

Here the writer would express his appreciation of Mr. Cooper's singularly simple notation, indicating, as it does, at once the load on the driver axles and the car load per linear foot possible by reason of his car load being one-tenth the load on each driver axle. Such a notation would be inapplicable in the foregoing series of loadings, wherein the deduced car load per foot is 11% the driver axle load, since the foundation of the base diagram is the car axle load, and not the load per linear foot. The series of typical loadings shown on Diagram No. 2 were framed for the writer's own use, and he hardly expected to find application for No. 1 333 or No. 667, as he considered Nos. 1 167, 1 000 and 833 representative of good practice in their respective places, that loading to be used which was 20% heavier than the weight of the actual trains in operation.

The writer draws a distinction between a "representative loading" and a "typical loading," meaning by the former a loading fairly close to the actual weights in use, but with their irregularities in loads and spaces rounded off or reduced to some uniformity; that is, the "representative load" is the one which truly and fairly represents the heaviest trains on the line. By the typical loading is meant one for which the bridging is designed, a load non-existing on the line, but to which limit it is supposed the actual or representative loading will tend, and in time reach.

The writer, in giving an account of his own work in the direction of a standard loading, does not thereby offer its results in opposition to Mr. Cooper's proposed loadings (from which it differs mainly in the train loadings), though he considers it rational to express the car load by axle concentrations when the engine load is so expressed. Though believing the ratio of tender to driver weight to be fairly represented by the fraction two-thirds, the variations in existing engines are too great to warrant objection to Mr. Cooper's ratio of six-tenths. And while deeming Mr. Cooper's 9 ft. between rear driver and first tender axle not borne out by the run of existing examples, and in the case of the heavier engines being from 25 to 30% too short, yet, as this is to

the advantage of the bridge, such lack of accordance with the actual machine is of no moment whatever.

The suggestion which the writer offers, however, is that Mr. Cooper's two concentrated weights be made uniform for the whole specification, viz.: Two loads, each 50 000 lbs. and spaced 7½ ft. apart, or 50% farther apart than the driver axle loads, for heavy passenger engines, of which such weights are representative, are as likely, by interchange, to run over a line whose bridges are of the class E.-25 as one whose bridges are of the class E.-35 or E.-40. The writer, not desiring to bring into his paper anything on those questions which Mr. Cooper in his article has expressly excluded from discussion, is, however, of opinion, based on experience, that bridge loading expressed by concentrated weights and definite spacing, extending even to the car load, is more appreciated and more intelligible to general managers and chief engineers than any equivalent uniformly distributed loading, or uniformly distributed load plus a concentrated weight, though the writer admits he strongly favors and supports the latter method in the present diversity, which exists in the several well-known specifications of railroads and bridge companies. The average general manager cares nothing about modes or methods of calculation; he is interested only in results. In building his bridges he merely wants to be assured by good authority that they are properly proportioned to carry the assumed loading.

The desire for unity forms the quest of philosophy. Reducing the whole field of bridge engineering to a common standard (and therefore a dead level) is not, however, unity of the kind sought. This sort of unity would change the engineer into a calculating machine; he is already quite enough of that. In engineering there should be no such thing as "Rome has spoken—it is settled." The writer holds it not only possible but likely that fully 25% of that which now forms the subject-matter of many "received" specifications will suffer material change at the hands of the coming generation of bridge engineers, even though the clauses in mind are now worded with all the certainty and "cocksureness" of a set of 39 articles, or of the utterances of certain professors of physical science and biology when sailing on foreign seas.

THEODORE COOPER, M. Am. Soc. C. E.—I have listened to the discussion this evening with much interest, but feel that some of the writers have failed to comprehend my position. I take the stand that we must have a standard train loading for our specifications, with which we can approach the managers of railroads when we wish to inform them in regard to the character of the bridge we propose to build for them. It will be a pictorial representation of what the bridge is to be capable of doing. In no other manner can we convey to the average railroad man the capacity of the structure; tables, diagrams,

concentrated loads, etc., will not do it. Of course, where railroads have their own bridge engineers, the selection of such a typical loading as I propose may not be necessary. Such engineers should be in direct touch with the management, and have their confidence. They can proceed in their own way to carry out the desired purpose. But I presume that the larger percentage of railroads in the United States build their bridges without the advisory action of a specially qualified bridge engineer. They simply ask the bridge companies for proposals upon some accepted specification, selecting with more or less judgment the kind of train load they desired used. It is for this class that general specifications are intended. I am not trying to convert the experienced engineer to depart from any methods of calculation which he may have determined to be sufficiently correct for his purposes. But it must be noticed that, in defending his methods, he is always anxious to prove that it only varies from certain standard train loads by certain small percentages, thus inferentially recognizing the necessity for a standard of final reference, an actual train load.

My specifications are in pretty general use throughout the United States, and I believe one feature that has been a satisfactory one is the series of engines from which any manager can select that engine best suited to his purpose. In preparing for a new edition I am willing to adopt a new series of train loadings, if we can settle upon a series that will be more generally accepted. The ones proposed in my paper appear to be far simpler than any that have been elsewhere proposed. The criticism made that they differ in certain features from any actual engines is true; but as these are intended for representative types solely for the purposes of proportioning our bridges, it is of no moment. It would be impossible to select a properly proportioned engine for the minimum size and then increase by regular gradations, as is proposed, without getting disproportionate results for the higher engines. It is necessary then, if we desire to retain the principle of increasing by percentages, to make a compromise in the proportional weights between the light and heavy engines. The weights on the pony wheels could be made less and those on the tender wheels more if found desirable, but the results would not be much different. As to the criticism that there is no necessary connection between the weight of the engine and the weight of the following train, this is correct; but, nevertheless, the tendency will always be in the direction of using heavy engines upon roads handling heavy cars. The great mass of bridges will receive their heaviest loads from two engines (all spans less than 120 ft.), whether the train is one car or fifty.

The remarks of Mr. Blakeley are very much to the point, and I am glad to have him bring out this point of the small increase of cost by the adoption of a heavier class of engine. It was my intention to have added some data in this direction, but I concluded it would be better if

brought out by an engineer of a bridge-building firm. It would have more weight with the purchasers of bridges to receive such a statement from the actual builders of such work than from an engineer. If this point was more generally appreciated, there would be fewer bridges built to the lighter specifications. What manager would hesitate to increase the strength of a 125-ft. span 20% for an additional cost of only $200? I hope the tendency of the discussion will be towards wiping out the lighter engines now used in our specifications. Personally I should like to exclude all engines and trains less than E.-30 of my paper.

J. E. GREINER, M. Am. Soc. C. E. (by letter).—In the present discussion it is hardly necessary for us to inquire very closely into the various causes which led up to the existing unreasonable conditions of train loads specified by railroads and bridge engineers, but in addition to the reasons advanced by the author we could perhaps add that the personal vanities of engineers have favored just such diverse conditions.

A maker of specifications tries to be original, or at least to be thought so, and therefore changes the usual permissible stresses, rearranges the clauses in some standard specifications, inserts a cranky idea of his own here and there, and designs an entirely new typical engine.

Whether or not the author can succeed in having a set of standard loads, such as proposed, generally adopted, will depend almost entirely upon the stand taken by those engineers who have already outlined a set of standards for themselves. They must be offered something as good or better than they now have, and must also be convinced of the advantages of such general standards.

We, undoubtedly, all agree with the author's general statements that refinements in dimensions or weights are useless, that the freight train loads are the only car loads necessary to consider, and that two types of engines can be made to represent the loadings produced by passenger and freight trains.

We must also admit that since train loads assumed for calculations of stresses are but typical, and that a change from the class now in use to a similar class with a slightly different distribution of loading and spacing would make practically no difference on our stress sheets.

The proposed types are very simple, agree closely with actual engines, and embrace the variation in weight which should meet all cases. The gradual increase in the weight of the loads by a definite percentage is a particularly good feature. This same plan has been followed on the Baltimore and Ohio Railroad during the last ten years, the 86-ton engines and tenders of 1884 having been increased first 10%, then 20%, and finally 25 per cent. Train loads increased in the same proportion.

The specified loading during the last five years, up to nine months ago, was 107.5-ton engine and tender, followed by 3 750 lbs. per foot. There are engines and tenders running over the road which weigh 107 tons. The specified loading was therefore recently increased to 125-ton engines and tenders, followed by 4 000 lbs. per foot. The wheel spacing and loading of this last typical engine are different from any engine known to the writer, and also bear no direct proportion to the old 86-ton typical engine; but having been used in the design of important bridges on the road, the writer did not think it worth while to make any change, and has therefore adopted this engine in the 1894 specifications, although he is not responsible for its design.

Should an understanding in regard to the author's proposed standards be reached, the writer, for one, will lend hearty co-operation, and would be perfectly willing to change his specifications accordingly.

The life of an iron railroad bridge has heretofore been scarcely 25 years, not because the bridge wore out under the traffic for which it was designed, but because the loading during that period has increased to such an extent that the older bridges, being generally of poor design, were no longer to be trusted. In 1869 the first 10-wheel camel engine, weighing 90 700 lbs., was run over the B. & O. R. R. In 1892, or about 23 years later, consolidation engines weighing 134 000 lbs. were brought into service. This shows an increase of 47.7% during the period. How much longer this increase will continue is a question on which railroad men differ. That there is a limit none can deny, and that this limit has not yet been reached is maintained by the best railroad men in the country.

Economical transportation is the great aim of railroad corporations; but as freight rates, through competition and other causes, are continually being forced down, and wages endeavoring to hold their place, something must be done outside of cutting wages to meet the downward tendency of rates. It is thought that this difficulty will be overcome by increasing the motive power, so that more tons of freight can be handled by the same train crew. So long as this will offset the low rates, we may expect the increase to be advocated; but we must bear in mind that heavier motive power means increased cost of maintenance, and that, after it has passed a certain limit, bridges which were perfectly safe under lighter loads will have to be strengthened or renewed at large outlays. The strength of existing bridges will, therefore, in many cases, determine the limit of the weight of our engines, as it is reasonable to suppose that the slightly reduced cost of transportation will hardly warrant the renewal of many bridges. There is no question as to this limit on the older railroads, but on new roads it will depend entirely on the economy of transportation.

Mr. Cooper's proposed E.-40 weighs 176 000 lbs. without tender. It

will probably be as heavy or heavier than any which will ever come into general use, but in the writer's opinion it would be a great waste to design structures for loads so much greater than are in service at the present time. A bridge built during the last five years by an intelligent and experienced designer, using proper materials and having first-class workmanship, should stand indefinitely under a train service even in excess of what it was figured for. In fact, the writer ventures to state that when the upward limit of engine weights becomes once definitely fixed, engineers will greatly increase their permissible stresses. Bridges are being designed for assumed typical engines, heavier than those in service; but even with this increase in view, we know that we have given the structure a large margin over and above what is necessary for our assumption.

So far as the train loading is concerned, the writer fully agrees with Mr. Cooper in the opinion that 4 000 lbs. per foot will not be exceeded except in isolated cases. In fact, the heaviest train load on many roads does not, and probably never will, exceed 3 000 or 3 200 lbs. per foot, except occasionally, and bridges which are perfectly safe for such loads will not be injured by an occasional heavier train load.

The writer can see no more necessity for proportioning all bridges, on branches as well as main lines, for the heaviest passenger engines than for the heaviest freight engines. Even if the heaviest engines should occasionally run over the branches, our bridges would be fully capable of carrying them. A large number of trains passing over a structure will wear it out sooner than heavier loads passing at longer intervals, as must be known to all railroad engineers having charge of bridges.

The writer knows of a number of bridges built at the same shops, during the same year and from the same plans, where one was placed on the main line and all the others on a branch. The bridge on the main line was racked to such an extent that it had to be trestled after 12 years' service. The others show no effects of wear, and are, to all appearances, just as good as when first built. The number of trains passing over the main line was probably three or four times greater than of those passing over the branch, although the weights of the engines were practically the same in both cases. It seems to the writer that, while such instances as these decidedly point out that bridges on main lines should be designed for the heaviest engines, they do not indicate any such necessity for branches having from one-quarter to one-third the traffic.

Joseph M. Wilson, M. Am. Soc. C. E. (by letter).—I quite agree with Mr. Cooper in his general remarks on the subject in question. To develop a certain rigid standard specification for the designing of bridges, complete in all its details, fixing absolutely by rule every item in the fundamental data, and then requiring certain exact methods to

be followed for the determination of every part of the structure, reduces the engineer to a mere machine which might as well be operated, as it were, from a central station by playing on a keyboard, and all originality, professional judgment and progress are lost.

It unfortunately happens that the system now almost universally adopted in this country, of having manufacturers prepare their own designs with their own engineers, has necessitated limiting the freedom of the engineer by minute specifications, which would not be the case if the engineer who designed the structure were an officer of the railroad company. The liberty of the engineer is therefore already more curtailed than wholesome for originality and progress. Under any circumstances, however, it is well in certain matters by "fair and thoughtful discussion" to approach uniformity of action, and Mr. Cooper's proposal to take up one subject at a time is a good one.

Perhaps no one is more important than train loading. The great difficulty with our older bridges was that not enough allowance was made in this item to allow for the rapid development and increase of loading which has taken place during the last 10 or 20 years; and while I am of the opinion, with Mr. Cooper, that the practical limit of loading has probably been nearly reached, still, I am in favor of a liberal allowance in this respect.

One of the important advantages of stone bridges lies in the fact of the great excess of strength over theoretical requirements, which, from the nature of the material and mode of construction, cannot be decreased. What I would like to see is a little of this in our metallic bridges.

Great refinements in dimensions and loading may, under such circumstances, well be omitted, as, for any material advance in the future, it is possible that the whole detailed distribution of engine loading might be changed, and an exact system adopted now would not suit then.

So far as the concentrated wheel method of computing bridges is concerned, it evolved very naturally in my own practice. The original rule followed in Pennsylvania Railroad practice was to use a live load of 2 000 lbs. per linear foot of track, which was in the year 1860 increased to 3 000 lbs. No account was taken of engine loading. About 1871–72, notice began to be taken of the increased concentrated loading on girders and other short spans from the driving wheels of engines, and a system of calculation was adopted providing for this, the concentrated wheel method being then practically started. In addition to this, the constantly increasing weights of engines of various types placed in service on the road and its branches required opinions from the Engineering Department from time to time as to the capacity of the bridges on these several lines to sustain such service. The question was not that of computing new bridges, where an assumed load

might be adopted, ample to cover all supposed traffic for years in advance, but of an opinion in reference to certain existing engines, and it became necessary to work up the exact conditions of the problem, a problem of constant occurrence on long railroad lines in active operation.

When a decision must be given on the reliability of an actual structure in use, under an established service, the bridge being perhaps already quite up to, or beyond, the limit of what might be called the standard, the concentrated wheel method must, in my opinion, still be used, and no general assumption will properly reach the case.

It naturally follows that, after using such a method for computing existing structures, it was adopted for new bridges also.

Then it was that typical engines were outlined, as representing the greatest development which would probably occur for some years ahead, and these were submitted to the Motive Power Department of the road for approval, and accepted.

In accordance with the practice at that time on the Pennsylvania Railroad, complete details of all bridges were prepared in the Bridge Department of the company, and no specifications were required for manufacturers, beyond those for material and workmanship. It was, therefore, some years before complete printed specifications were issued, giving methods of computation, the first being about 1882, and in the meantime an actual engine, Class M, had been put into service, the effects of which, under some conditions, were in excess of those from the typical engines, and this explains why, after such typical engines had been designed, a third engine was placed on the list.

It was supposed that ample allowance had been made for future increase for a long time to come, but actual engines now in use are already exceeding the loads of the typical engines then adopted.

The loading for our (Wilson Brothers & Co.) revised specifications of April 11th, 1891, used on the Philadelphia and Reading Terminal Railroad, is as follows:

Two typical engines coupled, weighing, with tender, 125 tons each, distributed as follows:

connected with a train weighing 4 000 lbs. per linear foot of track.

The "basis for discussion" offered by Mr. Cooper is certainly in the right direction, and while I am hardly prepared to say just now what modification, if any, I should propose in it, it meets with my approval in its general principles, and I should be glad to see substantial results follow its consideration.

HENRY B. SEAMAN, M. Am. Soc. C. E. (by letter).—In discussing Mr. Waddell's paper on "Some Disputed Points in Railway Bridge Designing," I ventured to suggest (*Transactions*, Vol. XXVI, p. 228), that the maximum concentration be 40 000 lbs., that the concentrations under the tender should be all alike, and should not be less than 23 000 lbs. each, and that the pony wheel of the engine should not exceed 16 000 lbs.

The engines outlined by Mr. Cooper seem very near the ideal, and his general proposition of change by percentage is an admirable one. It is to be regretted, however, that he found it necessary to use fractions of a foot in his wheel spacing, and in this respect the spacing suggested by Mr. Waddell seems preferable. The distance of 9 ft. between engine and tender is considerably more than that used in former typical engines, and is more than is often found in practice. The present tendency of new engines, however, is to increase this distance, and sometimes even to exceed that shown in the diagram. The drivers, on the other hand, are, in some instances, 4 ft. 6 ins. apart.

In adopting typical engines for new construction, we should not forget that they are not intended for examination of existing structures.

We are indebted to Mr. Cooper for this step toward a uniform practice, and particularly so since his discussions have not appeared to favor a movement of this kind.

WILLIAM BARCLAY PARSONS, M. Am. Soc. C. E. (by letter).—This paper marks a long step in advance in the matter of what shall constitute bridge loads, and, coming from Mr. Cooper, will probably be accepted more generally and more readily than if it had been written by any other advocate of the wheel load system. The great advantages that the proposed loadings offer are their great simplicity, freedom from small and senseless fractions of dimensions and a systematic increase in weights. But this simplicity openly abandons, in a large degree, the idea that the strains obtained by using wheel loads conform closely to the strains actually produced in practice.

The prime argument, therefore, urged against the use of a distributed load, with or without computations, that such a method gave results not strictly in accordance with those obtained by the use of wheel loads, can be urged with equal force against the loads suggested in this paper, which, too, will give strains differing from those found by using real or previously accepted typical engines. Of course, such accuracy is quite unattainable, and, even if it were, would be sadly modified by irregularity of construction, variations in adjustment, condition of track, inaccuracy of engine balance, and a host of other practical considerations. The difference, therefore, that separates those who, like myself, believe in equivalent distributed loads, and those who prefer wheel loads, has by this paper been much reduced.

In his verbal discussion of his own paper, Mr. Cooper gave an excellent reason for keeping typical engines in specifications, which was that managing non-professional officers pass on bridge construction who do not understand what an equivalent distributed load means. It may be necessary to keep up the typical engines for them, but that is not a reason for forcing them upon engineers.

If the corresponding equivalent distributed loads were computed and put in the specifications jointly with these typical engines, with their percentage of increase as proposed, an excellent and rational system would be obtained. Instead, however, of using two large axle loads, it might be well to use three, or a mogul design, to be employed in designing spans up to 20 ft. above which the distributed load is to apply.

Though I quite agree with Mr. Cooper that a standard specification is quite impossible and most undesirable, still I do think that a scale of bridge loadings to be generally adopted is possible, and Mr. Cooper by his paper has gone more than half way toward a ground where all can meet.

T. KENNARD THOMSON, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Cooper's paper cannot fail to please all bridge engineers and renew their hopes that a few standard engine diagrams will be adopted.

If the Society would appoint a committee to suggest two or three, or, at the outside, six, standard engine diagrams, I am confident that the majority of our railroads would at once make their selections therefrom. While our Society seems rather averse to making such suggestions, it would by so doing confer a great boon on many of its members.

As has been said, this question has been unfortunately confused with various methods of making the subsequent calculations. For while an engine diagram is as plain as a picture to the average railroad official, a table of varying uniform loads per foot is quite incomprehensible to many; and, besides, our grievance is not against the engine diagram pure and simple, but against the aggravating number and variation of them.

But if a few standards were selected, then each individual engineer could select what short cuts he desired, and, for that matter, complete tables of bending moments, shears, equivalent uniform loads, etc., would speedily be published and at the disposal of all. As an engineer who has been employed by both railroads and bridge builders, I endorse what Mr. Blakeley has said about our first-class railroads being willing to pay a few dollars more for the sake of getting a very much better bridge, which is only true economy. This, unfortunately, cannot be said of some of our second-class builders.

DAVID L. BARNES, M. Am. Soc. C. E. (by letter).—Mr. Cooper's

paper calls attention to the very important fact that the dimensions of freight cars are nearly at a maximum at the present time, owing to the vertical and lateral dimensions having reached the limits permissible within the right of way, and he argues that this fact gives rise to another of equal importance, namely, that the limit of weight per running foot of freight train will be practically reached when freight cars having the present maximum cross-section are loaded full with heavy grain. All that Mr. Cooper has said about the limits of weights and dimensions is easily shown to be true from an examination of the conditions under which freight cars are handled in this country. The practice of interchange of cars between different roads and the use of cars for a large variety of freight both tend toward the final adoption of a standard car that can be used, with reasonable economy, for both heavy, through and light local freight. Some enthusiasts who have traveled in foreign countries have been greatly impressed with what appeared to be a discovery of an error in American railroad practice, namely, the use of too large and too heavy cars for freight; but on making estimate of the effect of the use of the four-wheel light cars so common in Europe, it was found that the cars would be out of service most of the time and would be hauled light more often than loaded.

The next step in the reduction of the cost of car repairs and inspection must come from the adoption of a standard car that will be nearly the same in all particulars on all roads. This will so reduce the cost of repairs and facilitate the interchange of cars that most of the disputes in settlement and delays in transfer, now so common, will disappear. Steel under frames and steel in freight car bodies will not be generally adopted until there is some approach to a standard size, for the reason that the longer life of a steel freight car framing would prolong its existence to a period when standards will surely be adopted, and then it would be a great inconvenience to have undersized cars that are too good to be thrown away. The extra expenditure, about $100, for a metal car framing to get greater durability will not be made so long as there is no harmony in dimensions and details. The standard car will probably be made in two lengths, 36 and 40 ft., and will probably have about 75 sq. ft. of cross-section. Its capacity for a uniform distributed load will be about 100 000 to 120 000 lbs., with an empty weight of about 35 000 to 40 000 lbs. The flat and gondola cars will probably have the same under frames and will weigh somewhat less, as such cars have no roofs. Any one not familiar with the labors and trials of car inspectors and repairers at interchange points, can hardly realize the vast saving of labor that would result from the introduction of a uniform type of car details.

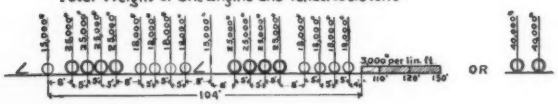
With locomotives as well as with cars the economical limit of weight will soon be reached. At present it requires a skilled man to fire the

THE COMPROMISE STANDARD SYSTEM OF LIVE LOADS FOR RAILWAY BRIDGES.

Sheet No. 1.

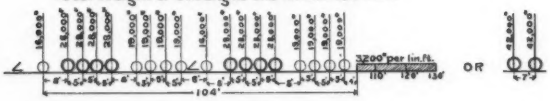
CLASS Z.

Total Weight of One Engine and Tender=93.5 Tons



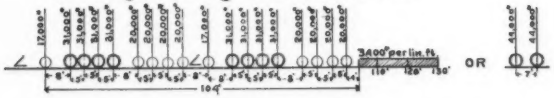
CLASS Y.

Total Weight of One Engine and Tender=102 Tons.



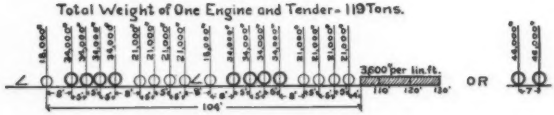
CLASS X.

Total Weight of One Engine and Tender=110.5 Tons.



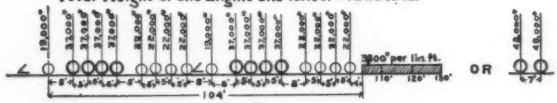
CLASS W.

Total Weight of One Engine and Tender=119 Tons.



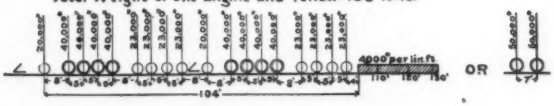
CLASS V.

Total Weight of One Engine and Tender=127.5 Tons.



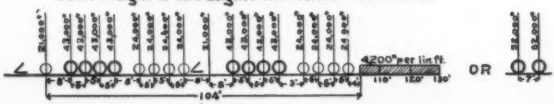
CLASS U.

Total Weight of One Engine and Tender=136 Tons.



CLASS T.

Total Weight of One Engine and Tender=144.5 Tons.



N.B. These concentrated loads are to be employed only as a standard of reference, and are the loads which were used in obtaining the data for plotting the curves on sheets Nos. 2, 3 & 4. For finding stresses use the equivalent loads given by said curves.

10 ft.-6 in. and 11-ft. fireboxes of large engines, and in the West, where the old type of deep, short and narrow fireboxes has been in use for a long time, the firemen grumble about the large grates. To fire a large grate a man must actually see the fire and place his coal where it is needed, while with the deep fireboxes the fuel rattles into place somewhat naturally. The principal economy of heavy engines and large cars is derived from the heavy trains of reasonable length that they permit; this gives a considerable reduction of labor per ton mile. But with the use of the quick-action air brake, which reduces the number of trainmen required, and the lower rates of labor that are clearly approaching, there will be less to be saved by hauling larger train loads than can be hauled with the present maximum weight of locomotives. The possible saving in fuel will be more attractive and remunerative, and the result will be a reduction of the rate of fuel consumption from 150 lbs. per square foot of grate per hour, the present average, to 90 or 100 lbs., which will give a large money saving, especially where coal is dear, as has been found in Mexico, where wages are low and fuel is high.

Altogether this paper by Mr. Cooper is very suggestive and points to one of the most productive fields of search for greater national economy, that of uniformity in design of such structures as are in common use.

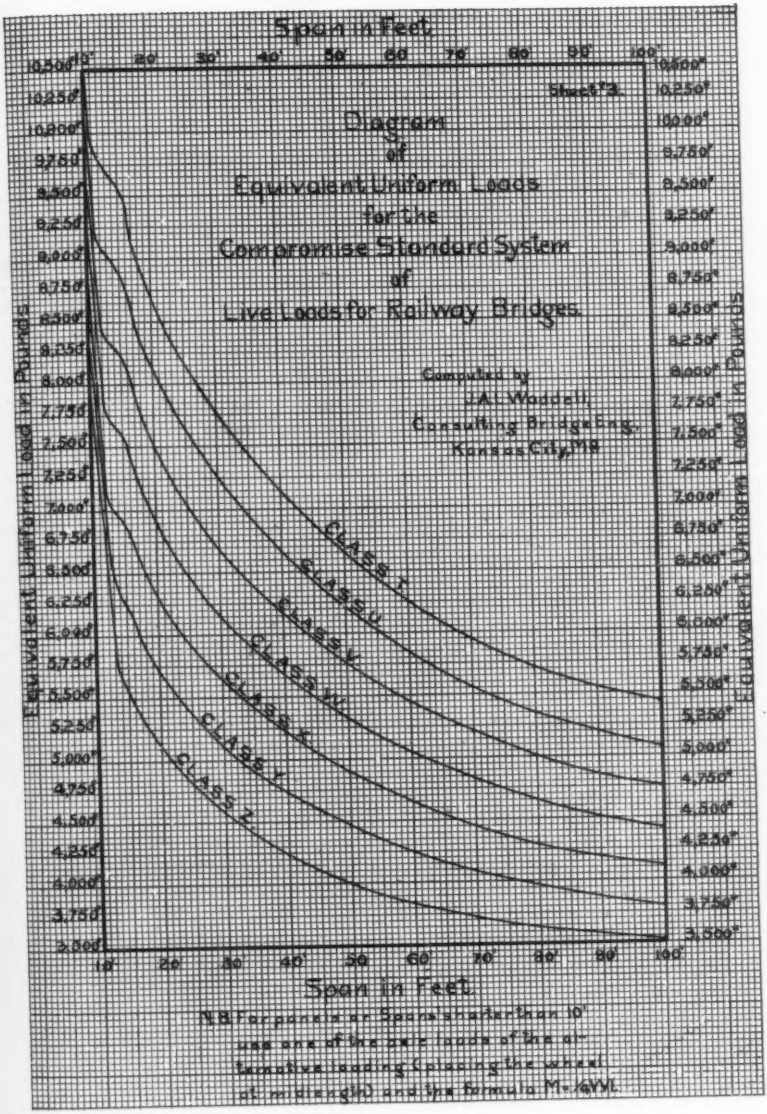
J. A. L. Waddell, M. Am. Soc. C. E. (by letter).—In discussing Mr. Cooper's paper there are two or three points upon which I wish to touch. The first is in respect to the advisability of establishing a uniformity in methods of bridge designing. While it is impracticable and fundamentally unadvisable to limit the designer in every petty detail, in that it would destroy individuality and retard progress, it is, on the other hand, greatly to be desired that some kind of system be established out of the chaos in which bridge designers are now laboring. Thus, for instance, if standard live loads, proper intensities of working stresses for all portions of bridges, the actual effects of impact, a correct practical column formula for steel compression members determined by experiments on full-sized struts, the best quality of materials from both an engineering and an economic standpoint, a standard for testing, and a standard of excellence for shopwork, were established, no possible harm could be done to the engineering profession; but on the contrary great good would result. Nor need the establishment of these standards be absolutely final; for they could be made, so to speak, to a certain extent elastic, so as to conform to future changes in loading of trains, improvements in manufacture of metal, etc. The tendency of the times is towards the systemization of technical knowledge; and, in my opinion, it is the duty of each member of our profession to do his share to aid in such systemization.

In treating of standard live loads for railway bridges at this late date, Mr. Cooper is a little behind the times; for nearly three years ago I brought the subject to the attention of the members of the American Society of Civil Engineers in a paper entitled "Some Disputed Points in Railway Bridge Designing," one of the principal of said points being the question of standard live loads. This paper was very thoroughly discussed by the members of the Society; and the discussion of the live load question has been continued, until a few months ago, in the technical press and by correspondence, the result of the entire discussion being a consensus of opinion, from which I have prepared and published in pamphlet form a "Compromise Standard System of Live Loads for Railway Bridges and the Equivalents for Same," and have distributed it very generally among those railway and bridge engineers who are, or who ought to be, interested in the subject, including every chief engineer of a railroad in North America whose address could be found.

This "Compromise System" differs materially from Mr. Cooper's proposed system in several particulars, as can be seen by referring to Plate XXIII; for while his heavy train load agrees very closely with "Class U" of the "Compromise System," his lightest loading differs essentially from that of "Class Z," in that his weights on tender axles and his car loads per linear foot are much smaller. By referring to Plate XVI, opposite page 90 of my paper on "Some Disputed Points," etc., it will be seen that the tender axle loads for my first proposed "Class Z" were 15 000 lbs., the same as Mr. Cooper has assumed for his lightest loading. Now, as it was due to the general consensus of opinion that I increased said tender loads to 18 000 lbs. per axle, it is evident that in this particular Mr. Cooper's lightest loading will not meet with the approval of the engineering profession.

Again, very few engineers can be found to-day who would countenance the use of a car load for bridges less than 3 000 lbs. per linear foot, knowing that it is possible to load some cars, as Mr. Cooper shows, as high as 4 000 lbs. per linear foot; consequently, Mr. Cooper's lightest car load of 2 500 lbs. per linear foot is decidedly too light.

Again, as it is generally conceded that the heaviest train loading should be about the same as that of "Class U" of the "Compromise Standard System," or that assumed by Mr. Cooper as his maximum; and as it is generally conceded that the lightest train loading should consist of two engines like those of the "Compromise Class Z" (or Mr. Cooper's proposed lightest engines, with heavier tender loads), followed by a car load of 3 000 lbs. per linear foot, it results that Mr. Cooper's proposed "multiple" or "proportional" system of live loads, although very pretty in theory, will not meet the requirements of practical men, especially general managers and chief engineers of railroads.


Span in Feet.
Sheet 3.
Diagram
of
Equivalent Uniform Loads
for the
Compromise Standard System
of
Live Loads for Railway Bridges.
Computed by
J.A.L. Waddell,
Consulting Bridge Eng.
Kansas City, Mo.
Class 1.
Class U
Class V
Class W
Class X
Class Y
Class Z
Equivalent Uniform Load in Pounds
Span in Feet.
N.B. For panels or Spans shorter than 10' use one of the axle loads of the alternative loading (placing the wheel at middle of) and the formula M=½WL

This "multiple" or "proportional" system, however, is not a new idea, for a year or more ago the same suggestion was made to me by a well-known professor of civil engineering. It was with regret that I declined to adopt it, my sole reason being that it would not meet the practical requirements of the case; because, while axle loads for modern engines vary from 25 000 to 40 000 lbs., maximum car loads per linear foot for the various railroads vary only from 3 000 to 4 000 lbs.

But, after all, does the proposed "Multiple System" possess any real advantage over the "Compromise Standard System"? An inspection of Plates XXIV and XXV will answer this question in the negative; for the curves thereon for all spans, excepting such short ones as are seldom adopted, vary from one another by regular gradations; consequently for any span length the equivalent live loads per linear foot increase in arithmetical progression, and therefore the weights of metal per linear foot for the various classes do approximately the same. The percentages of increase, however, for equivalent load and weight of metal per linear foot vary with the length of span. For instance, referring to Plate XXV, we find the following percentages of increase in load from one class to the next in reference to the loadings for "Class Z:"

100-ft. span	9.44%
200-ft. span	9.06%
300-ft. span	8.51%
400-ft. span	8.06%
500-ft. span	7.70%

With the diagrams of equivalent loads at hand, an engineer can make his interpolations and his "multiples" just as readily with the "Compromise Standard System" as he could with Mr. Cooper's proposed "Multiple System."

Just here it appears to me proper to say a few words more in respect to the "Compromise Standard System" and the "Equivalent Uniform Load Method," for the reason that both subjects were first brought up before the American Society of Civil Engineers in a paper presented thereto for discussion, and afterwards the discussion was continued by correspondence (subsequently published by the technical press), confined to the members of that Society, directly or indirectly interested in bridge building, until a general consensus of opinion was obtained.

After ascertaining that nine-tenths of the members of the Society interested in the subjects favored both the "Compromise Standard System" and the "Equivalent Uniform Load Method," some three or four months ago I addressed a circular to all the chief engineers of

railroads in North America whose addresses I could obtain, with a request that they vote on the following ballot:

BALLOT

On the Use of the "Compromise Standard System of Live Loads for Railway Bridges" and the "Equivalent Uniform Load Method."

Please cancel with ink either the words "DO NOT AGREE" or the word "AGREE," so that the remaining words shall indicate your intention.

I AGREE / DO NOT AGREE to use the "Compromise Standard System of Live Loads for Railway Bridges" when calling for bids on railway bridge work, or when having plans prepared for railway bridges.

I AGREE / DO NOT AGREE to specify that the "Equivalent Uniform Load Method" is to be used in computing stresses in the bridges that are to be designed for my road.

Signature of voter.

.....

Chief Engineer of the

.....

.....

(Please fill out the name or names of your road or roads.)

Up to date, not quite one-half of the parties addressed have been heard from, and the ballots continue to come in slowly; consequently, I shall not be able to announce the result of the ballot in the technical press for probably several months.

Of those heard from, about 4% objected to voting; 6% pleaded illness, want of time, etc.; 6% voted in the negative; 6% divided their votes, favoring either one of the questions, but not being prepared to vote on the other; 9% favored both questions, but did not wish to make any formal agreement, and 69% gave an unqualified vote in favor of using both the "Compromise Standard" and the "Equivalent Uniform Load Method."

PLATE XXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 692.
WADDELL ON TRAIN LOADINGS FOR BRIDGES.

In order to make the accompanying set of plates complete, I have inserted Plate XXVI, which gives at a glance the end shear from live load on any plate-girder span up to 100 ft. in length for all classes of the "Compromise Standard System."

Just here I desire to call attention again to the short method of finding live load stresses which I first proposed on page 265, Vol. XXVI, of the *Transactions*, and which I have elaborated in the lately published pamphlet before referred to, as follows:

"Where trusses have unequal panels or chords not parallel, the first step to take is the finding of all the dead load stresses by the graphical method, starting from one end of the span and working towards the middle, where the last stress is checked by the method of moments, and the correctness of the entire graphical work is thereby proven.

"The next step is to find from Plate IV the equivalent live load per lineal foot for the span, and therefrom the value of the panel truss live load L. Next set a slide rule for the ratio of dead load per lineal foot, and the equivalent live load per lineal foot for the span, and, by referring to the dead load stresses already found, read off from the rule all of the live load stresses in chords and inclined end posts.

"Next assume that there is an upward reaction at one end of the span equal to 1 000, 10 000 or 100 000 lbs. (according to the size of the bridge), caused by a load placed at the first panel point from the other end of the span; then find graphically the stress in each web member from end to end of span, caused by this assumed upward reaction. Then calculate the value of the live load reaction for the maximum stress in each web member by means of the slide rule and the following formula and table in which n is the number of panels in the span; n^1 is the number of the panel point at the head of the train, counting from the loaded end of the span, and C is the coefficient of $\frac{L}{n}$.

"Live load reaction for head of train at $n^1 = C \times \frac{L}{n}$.

n^1	C	n^1	C	n^1	C	n^1	C
1	1	7	28	13	91	19	190
2	3	8	36	14	105	20	210
3	6	9	45	15	120	21	231
4	10	10	55	16	136	22	253
5	15	11	66	17	153	23	276
6	21	12	78	18	171	24	300

"Then, still using the slide rule, find the greatest live load stress in each web member by the following equation:

Stress required = stress from assumed reaction $\times \frac{\text{actual reaction}}{\text{assumed reaction.}}$

"Where the panels are divided as in the Pettit truss, and where inclined subposts are employed, the tensile stress in the upper half of each main diagonal thus foundwill have to be corrected by subtracting therefrom a stress equal to $\frac{L}{2}$ sec. A, where A is the inclination

of the diagonal to the vertical. But when inclined subties are used instead of inclined subposts, the correction just referred to will apply only to the compressive stresses in the lower halves of the main diagonals. The reason for making this correction, as will be at once evident to any one who is accustomed to finding stresses in Pettit trusses, is that the method above outlined ignores the subdivision of the panels when ascertaining by graphics the stresses caused by the assumed upward reaction."

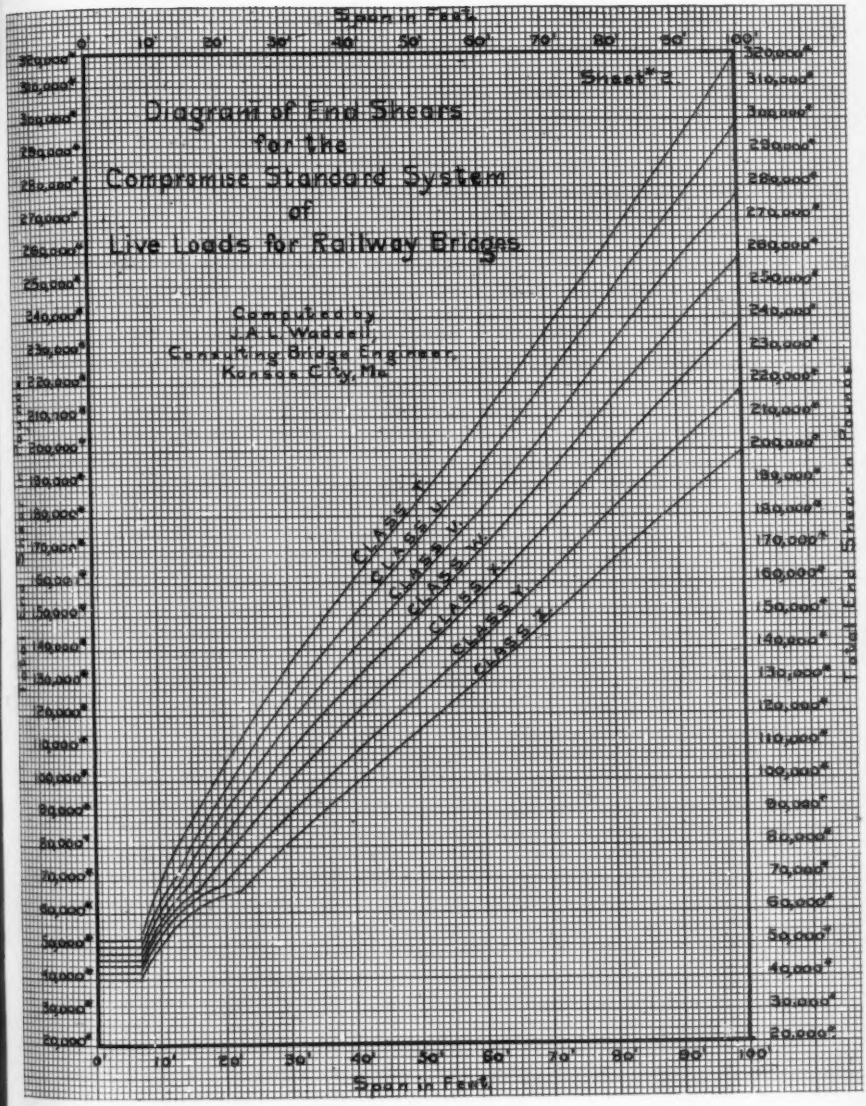
To illustrate the facility with which live load stress calculations are made in this manner by the "Equivalent Uniform Load Method," I would state that this morning I had occasion to compute all the live load stresses in the trusses of a double track, 624-ft. span in 16 panels (after having already found by graphics the dead load stresses), including both the tensile and compressive stresses on every main diagonal and post from end to end of span (caused by the advancing load), and the concentrations on hangers and primary inclined posts; and that it took me to do all this exactly 49 minutes by the watch. To find all of these stresses exactly by the concentrated wheel-load method would take a good computer from one to two days, provided that he ascertain for each truss member the exact position of the wheels to give the true maximum stress.

Again, a few days ago, using the method I am advocating, I figured in one day's work the total weight of metal in a span of 1 100 ft., computing the sectional areas of all truss members. It is true that I had at hand for reference a detailed design for a similar bridge of 1 040 ft. span. Unfortunately, I did not note how much time it took to calculate the live load stresses, but evidently it could not have been very long.

When one considers that stress calculations can be made thus quickly, and always with sufficient accuracy for all practical purposes, by the "Equivalent Uniform Load Method," he surely must come to the conclusion that the old, laborious and hair-splitting concentrated wheel-load method will soon become a thing of the past.

Mr. Cooper's remarks about the limiting weights of engines and cars are sound and very much to the point; for surely the practical limits of said loadings have been nearly, if not quite, reached already. In all of my investigations on the live load question during the last three years, I have failed to find a single actual train load as great as that of "Class U" of the "Compromise Standard System." By the way, I might here state that the extension of that system to "Class T" (which is to be used only for floor systems, primary truss members and plate-girder spans) was made to meet the theoretical but excessive requirements of one or two important lines of road, the bridge engineers of which appear to be endeavoring to discount the future.

Should any one desire to look up the discussion of this live load question since my paper on "Some Disputed Points," was issued,


Span in Feet.
Sheet № 2.
Diagram of End Shears
for the
Compromise Standard System
of
Live Loads for Railway Bridges
Computed by
J. A. L. Waddell,
Consulting Bridge Engineer,
Kansas City, Mo.
Class E
Class D
Class C
Class B
Class A
Class H
Class G
Pounds
Span in Feet.

I would refer him to the back files of the *Railroad Gazette* and the *Engineering Record.*

Mr. Cooper (by letter).—The writer is aware that some members of our profession do not believe in the use of train diagrams in bridge specifications; he not only believes it desirable to continue their use, but also thinks the general practice to be still in favor of that method of specification.

The purpose of the paper was to suggest a series of diagrams based upon the idea of regular gradations from the lightest to the heaviest engines; that the same could be discussed, to see if those who do believe in train loads could agree to some simple series, and thus do away with the many variations now in use.

Mr. Churchill's discussion is in favor of abolishing train loads, and however good his suggestion may be, it does not apply to the paper. The writer fully agrees with him in reference to the desirability of limiting, as far as we can, the standard train loads between 3 200 and 4 000 lbs. But it is beyond the power of any one to compel the use of any particular loading over all roads. Each of us may be able to exert the influence of our opinions over a limited range. It is necessary, therefore, if we desire to treat the matter generally, to make the series sufficiently elastic to cover a wider range than the above.

Mr. Fulton's suggestion to adopt one freight load of 4 000 lbs. is likewise impracticable.

The suggestions of Messrs. Blakeley and Bland as to increasing the relative weights of the tenders are pertinent, But the recommendation to also increase the load on the leading truck does not appear so desirable from the point of view of a typical train load, though it may be nearer actual conditions.

To express the car loads by definite spacings of the axle loads, as recommended by Mr. Bland, would make no difference in the final results, but might be confusing to those buyers of bridges who used cars of a different spacing.

The writer is glad to have the endorsement of such an able authority as Mr. Barnes upon his views in regard to the maximum loads of the future.

The writer regrets that he was unaware that the matter of standard live loads had been finally closed by Mr. Waddell. The writer's ignorance is perhaps due to the fact that he never knew scientific subjects could be settled by ballot. If the profession generally had known this important fact, how much worry and brain substance could have been saved.

It has, however, perhaps mistakenly, been thought that under some circumstances the opinion of one individual on a particular subject might outweigh the opinions of hundreds, and even millions, of other people.

Some years ago, while hearing evidence in a dispute between two large railroad corporations, a Mr. Blank, Chief Engineer of Blank Railroad, testified that he had been an engineer for 35 years, etc., and gave very strong evidence on one side. Upon cross-examination, it developed that the Blank Railroad was three miles long and only partially located; that he had never been connected with any other railroad; and that he had heretofore confined his talents to land surveying.

As Mr. Blank's name may still be among the list of chief engineers in North America, would his ballot on bridge subjects equal those of engineers who have made these subjects their life work?

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

693.

(Vol. XXXI.—February, 1894.)

PROTECTING PILES AGAINST THE "TEREDO NAVALIS" ON THE LOUISVILLE AND NASHVILLE RAILROAD COMPANY'S LINES.

By R. MONTFORT, M. Am. Soc. C. E.

READ NOVEMBER 1ST, 1893.

WITH DISCUSSION.

Protection by Concrete.—In the year 1880 the Louisville and Nashville Railroad Company acquired by lease the New Orleans and Mobile Railroad, running from New Orleans to Mobile, now known as the New Orleans Division of the Louisville and Nashville Railroad. With this lease it also acquired large creosoting works, located at West Pascagoula River. These works were then, and have since been, operated by Messrs. Putnam* and Tobias.

The New Orleans Division crosses a number of bays, bayous and rivers, all of which are spanned by means of yellow pine creosoted pile trestles, and iron bridges supported by creosoted pile piers.

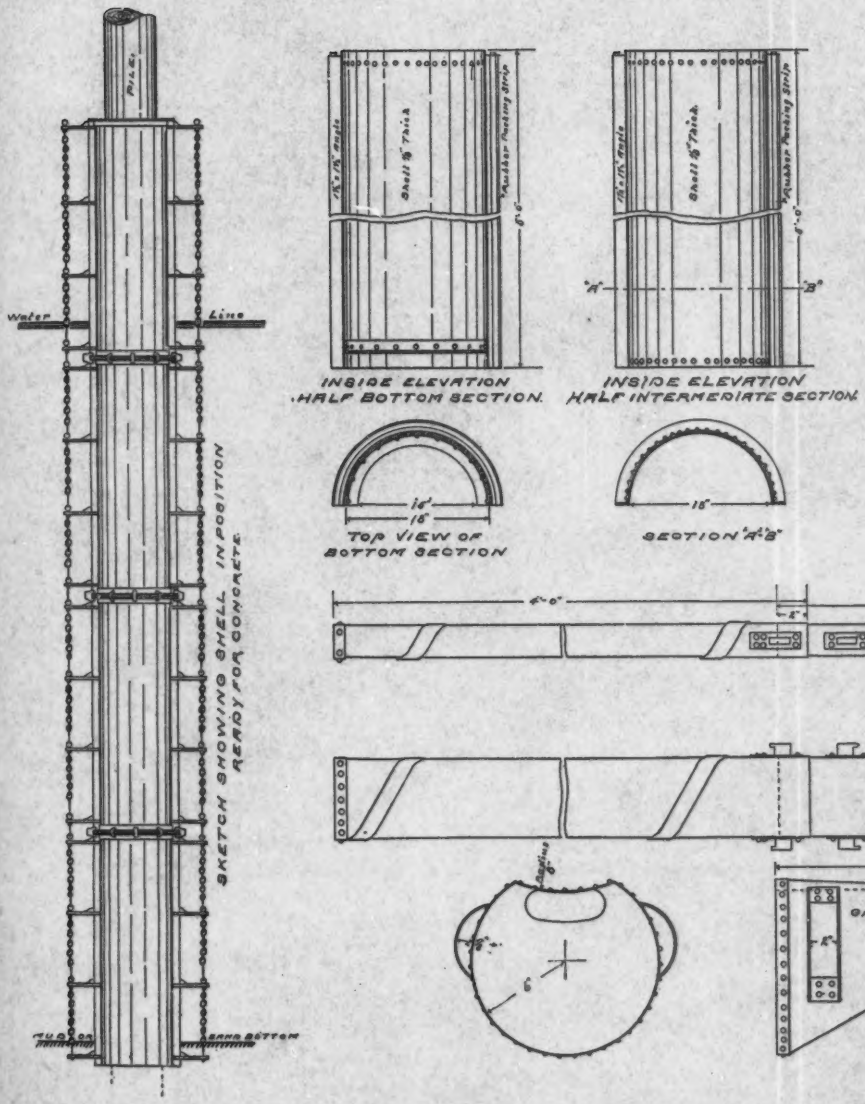
The total number of linear feet of trestle is 21 407, and of iron bridging 6 459. In considering the effect of the teredo on creosoted

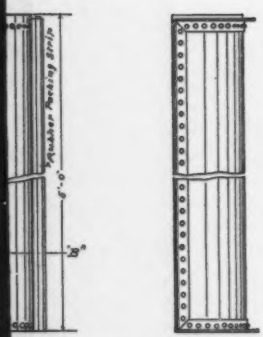
*J. W. Putnam, Assoc. Am. Soc. C. E. (recently deceased).

yellow pine timber, and the further means adopted to protect it against their ravages, the most noteworthy structures are Bay St. Louis and Biloxi Bay trestles. The bridge over the former is 10 241 ft. long, and consists of 10 050 ft. 2 ins. of creosoted pile trestle, with a 190-ft. 10-in. iron draw span; that over the latter is 6 040 ft. 10 ins. long, and consists of 5 850 ft. of creosoted pile trestle and a 190-ft. 10-in. iron draw span. These trestles were built by the New Orleans and Mobile Railroad Company in the years 1878–79, and replaced structures destroyed by the teredo, which abound in the waters of the Mexican Gulf and its estuaries, and are extraordinarily destructive on account of the long, warm season during which they work. It is no uncommon occurrence for the teredo to completely honeycomb an untreated yellow pine pile of from 12 to 15 ins. diameter in less than six months, so as to render it unsafe for structural purposes. In 1871 a serious accident occurred from this cause at Biloxi Bay Trestle, when a freight train went through the bridge, although the piles were only about ten months old; seven bents went down with the train. An examination showed that the piles were all eaten off close to the bottom of the water. When first built, Bay St. Louis Trestle had hardly been completed when it was found the untreated piles were so badly attacked by the teredo that it was necessary to commence rebuilding at once. In 1872 resort was had to covering the piles with copper before driving, and this gave better results; but the protection was not found to be perfect. It was, therefore, decided in 1876, to construct the creosote works already mentioned, and they were built at a cost of $60 000.

A close watch has always been maintained over the creosoted structures, in order to determine their behavior and economy as compared with similar structures of untreated timber. In addition to periodical inspections of those portions above water, the piles from the surface of the water to the top of the mud or sand, which composes the bottom, have been examined from time to time by divers. It was not, however, until 1886 that it was discovered the teredo had commenced its attacks on the creosoted piles. About the same time an inspection of the creosoted piles in the railroad company's wharves at Pensacola, Fla., which had been built in 1880, disclosed a similar condition of affairs.

In view of the vast amount of creosoted timber that existed in the structures on the New Orleans Division, already referred to, and also at Pensacola, Fla., and on the Pensacola and Atlantic Railroad,


Pile
Water Line
Mud
Sand Bottom
SKETCH SHOWING SHELL IN POSITION READY FOR CONCRETE
INSIDE ELEVATION HALF BOTTOM SECTION
INSIDE ELEVATION HALF INTERMEDIATE SECTION
TOP VIEW OF BOTTOM SECTION
SECTION "A-B"



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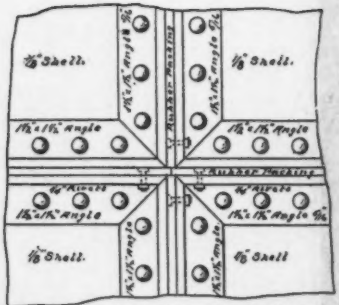
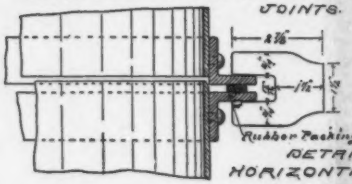


DETAIL of HORIZONTAL & VERTICAL
JOINTS.



DETAIL OF
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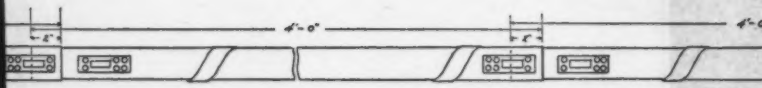
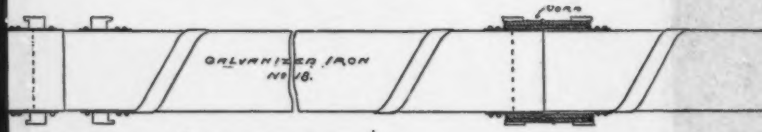
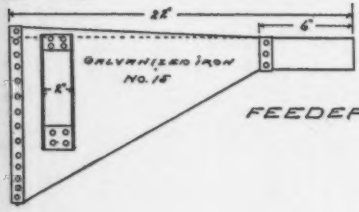
& CLAMP.



DUMPING PIPES.





FEEDER.

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USED IN PROTEC
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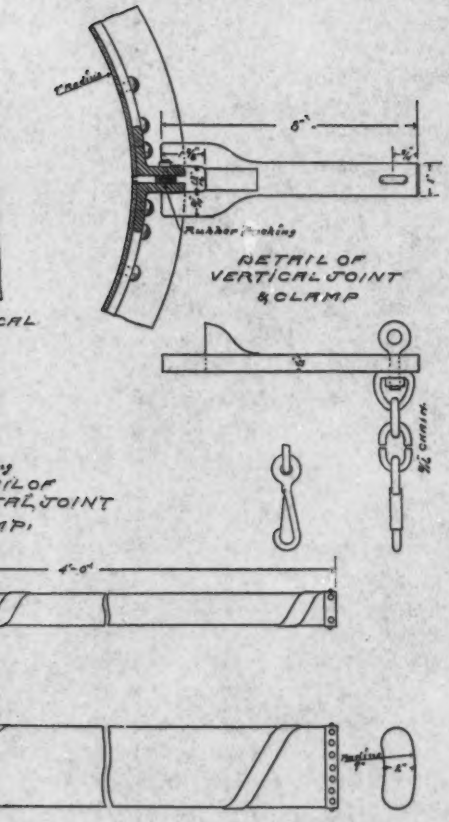


ROUGHT IRON SHELLS
ROTECTING PILES WITH CONCRETE
ON THE L.&N.R.R.

which was built in 1882–83, and is owned by the Louisville and Nashville Railroad Company, the question of determining, if possible, on some further means of protecting the creosoted piles against the teredo was of vital importance. In connection with the late F. W. Vaughan, M. Am. Soc. C. E., Consulting Engineer, the writer was instructed to investigate, experiment and report, with recommendations of what should be done. As a result, it was decided to adopt a thin coat of cement mortar or concrete, applied to the outside of the piles from the surface of the mud or sand at the bottom to the surface of high water at the top. In order to accomplish this, the work was conducted in the following manner : A shell of wrought iron, made in circular form, composed of several sections, each in two segments, so arranged as to be easily separated (see Plate XXVII), was placed around the pile ; the shell was clamped together above the water and lowered, one section after another, until it completely surrounded the pile from the surface of the water to a distance of from 6 ins. to 2 ft. below the bottom, varying with its hardness and the difficulty of forcing the shell down. A diver placed a pudding of "gumbo clay," inclosed in sacking, between the shell and the pile at the bottom before the shell was forced down, thus making the space between the pile and the shell almost water-tight, where the depth of water did not exceed 12 ft.; the water was then pumped out and the mortar, or concrete, poured in. Where the depth of water exceeded 12 ft., it was found impracticable to keep the water out, and the attempt to do so was given up. In such case the mortar, or concrete, was passed down to the bottom of the shell through a galvanized iron pipe of special shape (see Plate XXVII), so as to give as large an opening as possible without taking up too much space between the pile and the shell. A funnel or feeder was used at the top of the pipe, to render it easy to fill. The pipe was gradually raised as the concrete filled up the space between the pile and shell ; but the lower end of the pipe was kept constantly in the concrete, so as to prevent the concrete from falling through the water, which would separate its constituents, and, of course, ruin its adhesive power. The shell was allowed to remain three or four days, until the concrete had set, when the clamps were pulled off, and it was removed and placed on the next pile to be treated in a similar manner. Shells made of wood, held together with cast-iron bands, were also used extensively on account of their relative cheapness as compared with

the wrought-iron shell. In this way 2 638 piles were protected in the wharves at Pensacola, 840 in Escambia Bay Trestle, 413 in Biloxi Bay Trestle, and 216 in Bay St. Louis Trestle, making a total of 4 107 piles. It was only applied to piles that were found to be seriously attacked by the teredo, and that, if not protected, would soon be utterly destroyed. One important advantage which this means of protection possesses, is that it can be accomplished without in any way disturbing the piles or the superstructure which rests on them, and this was especially important in the case of Pensacola wharves on which large storage sheds had been built. If the piles had to be replaced by new piles, the floors and roofs of these buildings would have greatly interfered.

It is now seven years since the work of protection, as described, was begun, and although some little trouble has been experienced from logs and rafts striking the protected piles in rough weather and abrading or cracking the concrete on a few of them, the expense of repairs has been small, and all of the piles which have been protected are still in use, and likely to continue so for a number of years; whereas, if they had not been protected, it would have long since become necessary to replace them by new piles. Plates XXVIII and XXIX show piles broken off below the surface of the mud, and pulled up at Pensacola wharves three years after protection. They were found to be in good condition. Plate XXVIII shows a pile before the concrete was broken; Plate XXIX, after it was broken and the pile sawed in two. Not only did the concrete completely cover the piles, and was itself covered with oysters, barnacles, etc., but the cement had found its way into the teredo holes, filling them even to the heart of the pile, and forming within it perfect casts of the teredo.

To remove the concrete required several heavy blows with an axe.

The piles were finally split up into small pieces, without finding a single living teredo.

The concrete was composed of sand, gravel and "Alsen's" Portland cement, in the proportions of 1 of cement, 2 of sand and 3 of gravel; sufficient water was used to render it thin enough to readily pass through the pipe. When the concrete was broken off, many of the fractures passed through the centers of the flinty gravel, showing the great strength with which the cement adhered.

Pensacola Wharf.
Section of pile showing Old Teredo Holes,
Concrete protection covered with Barnacles, Oyster Shells
and Sea Weed.

The work of protection was first started at Pensacola, under the immediate charge of Superintendent E. O. Saltmarsh, to whom, as well as W. H. Courtenay, M. Am. Soc. C. E., Principal Assistant Engineer, a large part of the credit for the success of the undertaking belongs.

The cost of protecting piles in this way was found to vary from 80 cents per linear foot of concrete, measured on the pile, to $1 50 per linear foot, depending upon the length of the protection applied, the location of the pile, and the conditions of the weather ; the average was about $1 25 per linear foot. But as the number of feet that required protection—viz., from high water-mark to the surface of the bottom—was small compared with the total length of piles, the expense was much less than what it would have cost to have replaced the old piles with new creosoted piles.

Protection of Vitrified Clay Pipe.—Last year, 1892, a careful surface inspection and inspection by diver showed the following condition of the piles at Bay St. Louis and Biloxi Bay trestles :

	Bay St. Louis. Number piles.	Biloxi Bay. Number piles.	Total Number piles.
Not attacked	618	334	952
Few small holes	225	52	277
Full of small holes	567	273	840
Large holes	997	576	1 573
Piles already protected by concrete	216	413	629
*Decayed	3	....	3
Totals ...	2 626	1 648	4 274

* Replaced by new piles. Probably not sound when treated at the time trestle was built.

From the above it will be seen that a very large number of piles in both of the trestles needed immediate protection, and, as the caps and stringers were too light for existing rolling loads, it was decided to strengthen them at the same time that the piles were being protected, and to make a complete job by protecting all of the piles not already protected by concrete.

These trestles have been in service for 14 years, yet the timber was found to be in a good state of preservation from decay, with the exception of a few ties, which were replaced by new cypress ties (see Plate XXX). It is expected that the life of the new cypress will be as

long as the remaining life of the old creosoted pine above the water-level, and that when renewal above that level is required—10 or 12 years hence—a frame trestle can be placed on top of the protected piles, as indicated on the drawing.

It will be noticed, by reference to the drawing, that, instead of using concrete protection on these two trestles, vitrified clay pipe was substituted for it. The reason for this was two-fold—first, on account of economy; secondly, because it was believed that the pipe would be a more certain protection to the pile immediately above the surface of the mud, it having been found, in a few cases, that the mud had become mixed with the bottom of the concrete, and, of course, killed the cohesive power of the cement, thus leaving 2 or 3 ins. of the pile above mud line exposed to the attacks of the teredo. In most localities this is not of vital importance, as the teredo usually works near the surface of the water and between high and low tide, his ravages becoming less as the bottom is approached. But at Biloxi the case is reversed, due to fresh water flowing on top of the more dense salt water, hence the teredo thrives better near the bottom, and it is quite common to find piles there entirely eaten off at the surface of the mud, while showing no indication of having been attacked at the water-surface or between high and low tide. To this may be attributed the accident of 1871, already referred to. The same is true to a less extent at Bay St. Louis.

In order to allow the sections of pipe to be readily passed over the tops of the piles, it was desirable to cut the piles off at a sufficient distance below the stringers. This necessitated the use of a bolster, or some other contrivance, to support the stringers, and, as it was also necessary to strengthen them, the bolster was made to extend the full panel length and its upper surface keyed to the lower surface of the stringers, with cast-iron keys, so as to make it act as far as possible in conjunction with the stringers. This arrangement also permits the easy removal of either of the sticks composing the stringer, or of the bolster, independent of each other, in case any one of them should decay before the others.

A great deal of trouble had been experienced at both of these trestles, on account of fires caused by sparks dropped from the engines of passing trains during droughts, and it was therefore determined to box in the floor and cover it with gravel.

Pensacola Wharf.
End view of pile protected with concrete,
Pile removed three years after protection.

The sections of pipe were fastened together with a composition of pitch and sand, applied hot. As each joint was lowered below the surface of the water, the composition was cooled by the water and became hard and held the two sections of pipe firmly together. A third section was then added, and so on, until the pipe reached from a solid bearing, 1 or 2 ft. below the mud bottom, to high water-mark. Jacks were used to force it down. The space between the pipe and pile was then filled with sand, which was allowed to settle. Finally, after several months, the pipes were again refilled with sand, to make up for settlement and that which waves had washed out, and were then sealed on top with the same composition used for cementing the joints of the pipe.

The cost of protecting piles by means of pipe was found to be 66 cents per foot of pipe where 16-in. pipe was used, and 71 cents per foot where 18-in. pipe was required. These figures represent everything connected with the work, including freight on pipe from St. Louis, Mo., to bridge site—quite a large item. The length of pipe used on each pile varied from 15 ft. at the centers of the bays to 2 ft. 6 ins. near the shores. As the piles were from 50 to 60 ft. long, the cost of protection was much less than would have been the cost of renewal with creosoted piles.

A test pile, known to be infested with the teredo in a live and healthy state, was treated in this way by having a pipe placed around it, without filling with sand. At the end of 24 hours it was found that the teredos were all dead and turning black. At the end of an additional 48 hours the pile was again examined, and the bodies of the teredos had almost disappeared. At the end of an additional week it was again pulled up, and split into pieces, and nothing could be found of the teredos, except the holes that they had left ; all parts of their bodies had disappeared. With the space between the pipe and pile filled with sand, we can therefore feel assured of the utter destruction of the teredo and the permanent protection of the pile from their further ravages. The result of the experiment might fairly be inferred from the fact that the teredo never attacks a pile below the surface of the mud, or sand, into which it has been driven.

In cutting off the 4 274 piles at both trestles, but three were found to be decayed, which, considering their age (14 years), is quite a good record for yellow pine timber treated with dead oil creosote. From

all indications they will be good for at least 10 years' further service. Untreated yellow pine piles would not last in this climate more than an average of seven years, assuming that the teredo did not eat them up, as it would in far less time.

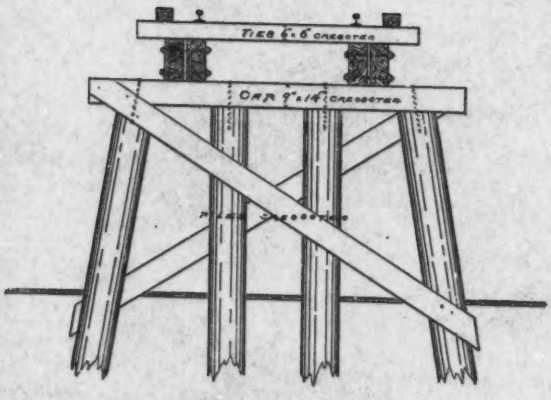
* * * * * * * * *

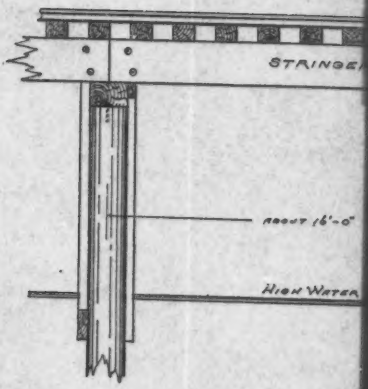
Since the foregoing was written, the Gulf coast was visited by the most disastrous hurricane known in its history, which caused the death of some 2 000 people and inestimable damage to all kinds of property. The Louisville and Nashville Railroad Company was by no means exempt. An idea of the force of the hurricane can be gained by noting the damage all along the line from Flomaton, Ala., to New Orleans, La., a distance of 202 miles. For that distance immense numbers of trees, many measuring over 2 ft. in diameter at the butt, were twisted off and blown down across the track. Telegraph poles and telegraph wires were also blown down.

From Bayou Sara Bridge, nine miles north of Mobile, to Mobile, the track was covered with heavy drift. Trees of large dimensions and saw-mill logs, 60 to 70 ft. long and 30 ins. square at the butt, were piled one on top of the other to a depth of from 4 to 8 ft. The track was also washed off the road in some places. Chickasawbogue Draw Bridge was moved bodily 6 ins. out of line. In Mobile the water was 4 ft. deep in the Union Depot, and the same depth on the floor of the Cotton Exchange. Large flat boats were stranded in the middle of the streets, and many dead bodies were floating in the river. Fortunately there was only one train between Bayou Sara Bridge and Mobile ; it was side-tracked at Magazine Point and was protected from the storm. None of the crew lost their lives, but a section-man in the same locality was not so fortunate; he was carried off the roadbed and drowned.

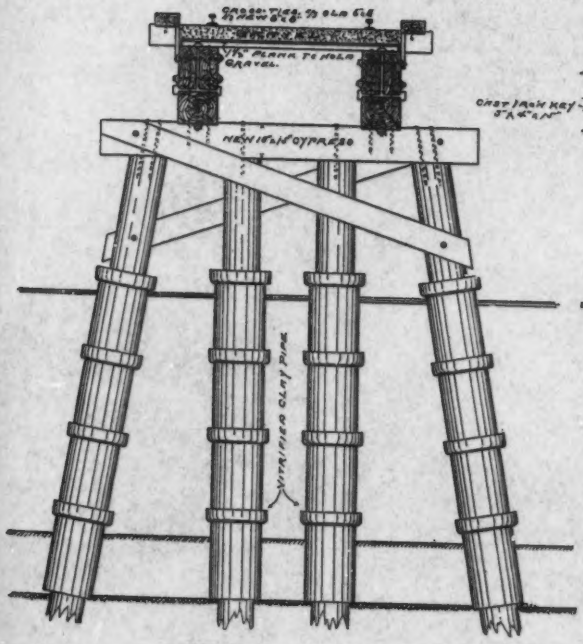
The New Orleans and Mobile Division was a much heavier sufferer. From East Pascagoula Bridge, at Scranton, to West Pascagoula Bridge, at West Pascagoula, a distance of three miles, the entire track—rails, fastenings and ties—was carried off the roadbed and into the marsh, two to three miles away. The roadbed at this point is about 10 ft. above ordinary high tide, and no serious trouble had ever been experienced with it from the time the road was built until October 2d, 1893. Three large schooners were landed on the embankment, one in the center of the

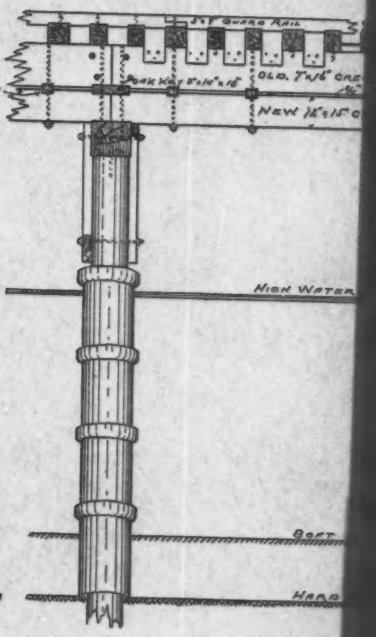
OLD TRESTLE

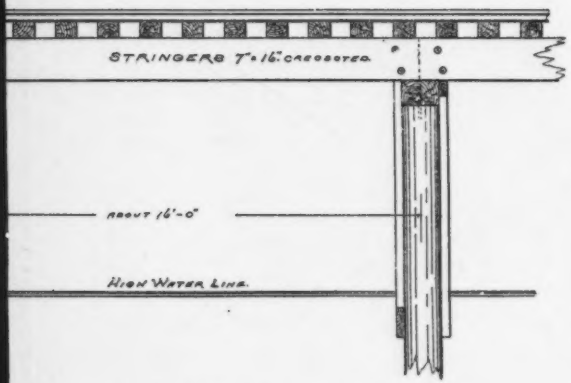




TRESTLE WITH DECK STRENGTHENER & PILES PROTE







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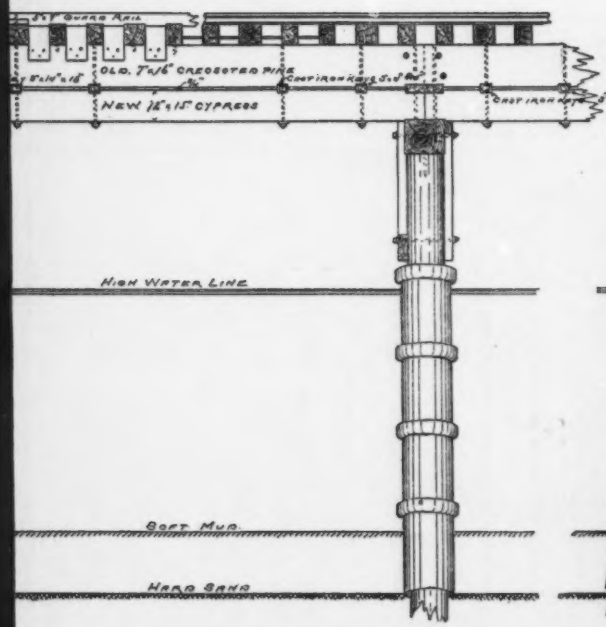
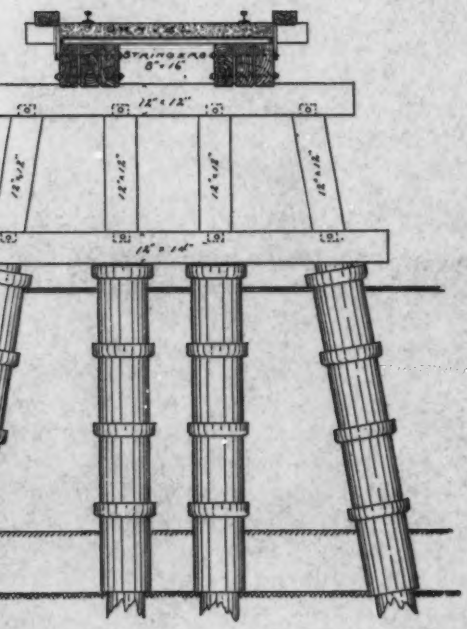


PLATE XXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 693.
MONTFORT ON PILE PROTECTION.

METHOD OF STRENGTHENING AND PROTECTING TRESTLES ON THE N. O. & M. DIVISION OF THE L. & N. R. R.

PROPOSED METHOD OF PLACING FRAMED BENT ON OLD PILES WHEN SUPERSTRUCTURE NEEDS RENEWAL.



roadbed, the other two about half way up the bank, which was badly washed by the waves.

At East Pascagoula Bridge one end brace of a 77-ft. iron span was bent so badly as to render the structure unsafe. The floor of the entire bridge, 700 ft. long, was thrown out of line; wrought-iron tension members of all of the spans were bent and kinked, but not to a sufficient extent to render them dangerous or to make repairs difficult or expensive. At West Pascagoula one 97-ft. span was completely wrecked; the iron phœnix column posts were broken to pieces, and the superstructure, with exception of the floor, had dropped into the river. The floor was hanging to the rail, which did not break, but sagged down about 6 ft. The floors of the other spans of West Pascagoula, 1 350 ft. in length, were in bad shape—knocked out of line and up against the trusses by the force of the waves and drift.

At both ends of Biloxi Trestle the embankment was washed away, as was also the floor of the trestle, with the exception of 975 ft. at the north end, which was intact; the remainder, about 4 875 ft., including the caps, stringers, ties and rail, was gone. All that part south of the draw was afterwards found from one to three miles up the bay; the material north of the draw was found several miles down the bay.

The hurricane first blew the water into the bay, and afterwards wheeled around and blew it out again.

The draw-tender, Mr. Vanderpool, and his assistant, were in their house at the draw at the time the storm came up and remained there until about 6 A. M., when they became frightened and attempted to escape by means of the trestle, which was not damaged up to that time. The assistant draw-tender had only proceeded about 20 ft. when a wave struck him and carried him away. His body has not yet been recovered. Vanderpool, the draw-tender, then abandoned the attempt to escape and climbed up into the trusses of the draw span and tied himself on with a rope. While in this position he had full opportunity of observing the effects of the storm, which did not reach its maximum for some time after he and his assistant had attempted to go on shore. He states that the waves rose many feet above the top of the trestle and the floor of the draw, and that boats were carried clear over the trestle on these waves. The bay was full of drift, the accumulation of many years in the marshes. This drift consisted of immense trees, which struck the trestle and bridge with enormous force,

battering the piles and caps, and finally, by continued pounding, as each wave struck them, knocked the caps loose from the piles, when the floor of the trestle was carried away. This allowed the drift to escape, and it finally went to sea. The condition of the trestle, after the storm, fully confirms Mr. Vanderpool's statements. The tops of the piles, as well as the caps, showed the heavy blows to which they were subjected by the drift striking them. Seventy-seven piles were entirely broken off so they cannot be used again; many of them were carried away and are nowhere visible. Other piles, where the drift bolts did not pull out, were burst in two.

The caps were secured to the piles in the usual way, by means of drift bolts. An additional drift bolt was, however, used in the two outside piles, making a total of six drift bolts, ¾ in. in diameter and 24 ins. long, to each bent.

When the floors of Bay St. Louis and Biloxi trestles were strengthened, the grade of the track was raised in low places, in order to remove depressions which previously existed in the track. No change was made in the method of securing the caps to the piles, as prior to the hurricane of October 2d no trouble had ever been experienced with caps similarly secured. Had extraordinary means of securing the caps to the piles been adopted, such as wrought-iron straps fastened by cross-bolts through the piles, it is very doubtful whether the trestle would not have been in a worse plight than it was; in other words, it is more than probable a very much larger number of piles would have been broken off and carried away, had not the detachment of the floor liberated the drift which was pressing against the piles and which broke 77 of them. In this connection it is worthy of note that a large proportion of the broken piles that are visible have the caps still attached to them. Had the piling at Biloxi and Bay St. Louis trestles been lost, it would have taken months before the road would be open for through traffic.

The floor of the draw-tenders' house was destroyed and the whole building badly damaged. This building is supported on piles and is level with the draw. The floor of the draw was also damaged, and a number of the floor-beams carried away, while others were detached from their places. These floor-beams were large sticks of timber 8 x 16 ins. by 17 ft. long, resting on the wrought-iron bottom chord of the truss, and were secured by two bolts 1¼ ins. diameter. The force

with which they were struck must therefore have been tremendous. The lateral bracing between the lower chords of the draw was also broken in several places, and the center cast pedestal, on which the weight of the draw rests, was cracked.

The tops of the pile piers, which supported the ends of the draw, were carried away.

At Bay St. Louis Bridge the embankment at the north end was washed away. The floor of the trestle for 7 170 ft. was carried off in a similar manner to Biloxi, leaving 2 880 ft. at the south end of the trestle intact. A schooner had crushed through the trestle, carrying with it about 48 ft. of the floor, as well as the piles supporting it. The drift at Bay St. Louis was tremendous. One saw mill alone is reported to have lost 10 000 logs, all of which went through the trestle and out to sea. In addition to saw-mill logs, there were acres of drift from the marshes, composed of immense trees such as those already described, and one of these, over 60 ft. long, still remains tangled up in the piles. All of the bathing houses, both at Biloxi and Bay St. Louis, were carried away early in the storm, as well as the timber piers leading to them; they helped to add to the accumulation of drift against the trestle.

The drawbridge at Bay St. Louis was found in a similar condition to Biloxi drawbridge, already described; many of the floor-beams gone and others knocked loose and up against the trusses.

The draw-tender and his assistant at Bay St. Louis were, however, more fortunate than those at Biloxi. They became alarmed earlier and succeeded in escaping by crawling and holding on to the rails when the waves broke over them.

The floor of Bay St. Louis Trestle was carried out to sea. A large part of it was stranded on Ship Island, some 15 or 16 miles distant, and this it is expected to recover.

Forty-five piles in Bay St. Louis Trestle were gone or completely broken, making a total for both trestles of 122 piles destroyed out of a total number of piles in both trestles of 4 274: that is, the number destroyed was a little less than 3% of the total number of piles in the trestles. The remainder of the piles can be used again, although many of them are badly battered and split.

The pile protections at both trestles suffered to some extent from being struck with the drift, but the damage is much less than one

would expect. The amount of pipe which will have to be provided to make repairs is only 5% of the total amount of pipe originally used in protecting the piles, and this includes protection of the new piles used in replacing those that were entirely destroyed.

The repairs that have been made to the trestles since the hurricane are of a temporary character, sufficient to permit of the resumption of traffic. When permanent floors are again placed on them, the grade of track will be further raised in the lowest parts about 8 ins., which can be done at slight expense. The draws will be raised a similar amount. The caps will be anchored to the piles with wrought-iron straps and bolts, in addition to the drift bolts, and will be cut off flush with the face of the outside battered piles, so that it will be impossible for drift to catch under the caps.

Extra heavy bolts will be used for securing and binding the stringers, caps and ties together, so as to better provide for storms hereafter; but even with all these precautions, if a hurricane such as that of October 2d should again occur, there is no assurance that any trestle which it is practicable to build will stand the strain.

South of Bay St. Louis, eight miles of track were carried entirely off the roadbed, together with the sand ballast. In some places the marsh had floated over the roadbed, and when the water went down left a deposit of from 3 to 4 ft. in depth.

At Pearl River the iron bridge was in bad condition; trusses were about 2½ ft. out of line, and the floor more than that. The draw-tender stated that at one time during the storm a pile pier, which supports two of the spans, was 10 ft. out of line, but after the wind turned and the drift was carried away, it came back to within 2½ ft. of its original position. Two of the piles in this pier, each about 18 ins. in diameter, were entirely broken off; the iron top had been lifted off the piles and knocked up stream. Many of the truss-bars were bent, and the whole floor was out of place.

The loss of life in and around Pearl River was very great.

The work of repairs was commenced immediately after the storm, and the road was open for through traffic on October 17th.

Diligent inquiry among the older citizens and fishermen residing in the vicinities of Bay St. Louis and Biloxi, as to when high water or a hurricane similar to that of October 2d had previously occurred, developed the fact that in 1860, and also in 1852, the water had been as

high, if not higher; but on both of these occasions the rise was gradual, occupying from three to four days; whereas, in the recent hurricane the water rose in a few hours. No one recollected so severe a hurricane as that of October 2d, 1893, and it is certain none ever occurred that caused the same amount of damage and loss of life.

DISCUSSION.

GEORGE H. THOMSON, M. Am. Soc. C. E.—This paper is interesting to me for the reason that while Resident Engineer at Bay St. Louis in 1869 the building of this bridge was begun. The Mobile line running from New Orleans to Mobile, 140½ miles, was located by the late Henry Van Vleck, a pupil of the late John B. Jervis, M. Am. Soc. C. E., and is noted for its long tangents, its easy gradients, its 30 miles of marshes and 6 miles of bridges mostly subject to the sea worm. The temporary character of the bridges was incidental to hasty construction. In 1869 a pile in salt water with worms was thought to be efficient for three years.

Taking the Bay St. Louis bridge for an example, the water in the bay was fresh up to June 1st, owing to the high water in Bayou Jourdan due to a rainy spring; the worms were not active that year. In the year 1870 the worms were at work, and the specimen of this piece of riddled pile is the work of about, say, 90 days of teredo. None of the holes much exceed $\frac{3}{16}$ in. diameter, yet the piece is fairly honey-combed, and is now of no structural value.

The first Bay St. Louis bridge was a trestle of 16 ft. bents, resting on five piles per bent; the rails for two-thirds of its length were about 7½ ft. above ordinary water.

When the worms (in 1870) began to attack the bridge, the railroad company began the protection of piles about in this way: The piles, for the water length, were worked down to about 12 ins. diameter; then covered with a course of felt; then yellow metal covering was applied, the seams being closely nailed. The protection of Biloxi Bay piling was done with zinc instead of yellow metal. The zinc covering in about one year began to fail by corrosion; the yellow metal was more successful. Other attempts at protection were tested. Piles were charred for the water length of pile and then while hot were soaked with dead oil; then, when cold, black varnish was applied to "hold" the charring firm under the jar of pile hammer. I heard some years after that this work was a success so far as the protection from the

worm, but the piles' heart rotted above the water-line in about six years.

I have seen one worm only that had a diameter of $\frac{5}{8}$ in.

A boat infested with worms will sometimes get rid of them by a two weeks' sojourn in fresh water; this does not always result in the destruction of the worm. Old residents say that worms born in the vicinity of bayous are killed by "tying up" or immersion in fresh water. Very salty water and a hot season find energetic worms, and three months is long enough to fairly riddle an 18-in. pile in some situations.

JAMES DUANE, M. Am. Soc. C. E.—The teredo in the Gulf water seems to accomplish in three months what ours here will in two years; it is a question whether this is due to the temperature of the water or the size of the worm. Probably they vary with the kind of timber they frequent, but in the cases I know of they have been very small, perhaps not over $\frac{1}{16}$ to $\frac{1}{8}$ in. in diameter and $\frac{3}{4}$ to $1\frac{1}{4}$ ins. long; we did not find that the depth of water made much difference. A heavy oak timber box covering the water pipe laid to supply North Brother Island was eaten away in two years; it may have been partially destroyed much earlier than that; the break occurred in 65 to 70 ft. of water.

GEORGE HILL, Assoc. M. Am. Soc. C. E.—Speaking of the size of the worm as affected by the warmth of the water, ten years ago I knew of some worms in the south branch of the Shrewsbury that attained a diameter close on $\frac{1}{2}$ in.; they went through a pile 8 ins. in diameter, driven to hold the oyster boxes in place, the water not exceeding 4 ft. in depth in about 18 months.

In connection with the life of the pile, Mr. Thomson speaks of the charring as leading to the rotting of the interior of the pile. Mr. William Kennish, connected with the Carolina Oil and Creosote Company, published something relating to charring under high pressure. He illustrated in *Engineering News* a section of pile that had been treated with creosote and some that had been charred first and then treated with creosote. Those treated with creosote showed patches where the worms had gone into them very seriously, while those that had been first charred and treated under pressure were perfectly sound. The explanation was that the charring removed some of the inner portion of the bark, which interfered with the proper impregnation of the pile. It occurred to me that perhaps that was the explanation of the deterioration of the piles at Bay St. Louis and at Biloxi. As I have always labored under the impression that a properly creosoted pile was indestructible, the discussion has been interesting to me, as raising the question of what caused it, and why they did not act in accordance with our other experience in the matter. If Mr. Thomson has had any experience in that particular line, I would be much interested to know about it.

Mr. THOMSON.—In regard to the charring of piles, a man in Texas first suggested the use of charring. He tried them, and, so far as he could see, they were successful. The piles rotted inside, above the water, in six years, as they would anyway.

I tried from 1869 to 1872 all sorts of patents for the protection of timber. The worms seemed to attack the patent timber sooner than the natural, with one exception. In 1866 I was a student of chemistry with Dr. George Hadley, of Buffalo. He told me that he had spent a part of his youth in Bay St. Louis. He sent me a piece of his treated timber in 1869, which, up to January, 1873, had not been touched by worms.

The paper describes a great storm. In 1869 there occurred just such a storm at Bay St. Louis. The sea was at first very calm, but the sky was dark, and a low rumbling noise could be heard as the storm approached the shore. The water raised 7½ ft., the trees were uprooted, all the bathhouses washed away, and the leaves, 500 ft. from the shore, stripped from the trees. Some of the old darkies told me that their recollection was that these storms came after every other epidemic of yellow fever; the yellow fever came every 14 years, the storms every 28 years. After this storm I thought the grade of the bridge should be raised.

I think in the present state of the iron market we could put in permanent bridges and maintain them a little cheaper than is done now.

A. McC. PARKER, M. Am. Soc. C. E.—In 1871, the teredo was very plentiful in the waters of this harbor, and the life of piling in the clear water near the pier heads was not over about 10 years. At that time all of the sewers discharged at the bulkheads, and, while the water close in shore was foul, but little of the sewage was carried out near the heads of the piers. The work of extending the sewers in boxes under the piers was begun about 1872, and at this present time nearly all of the sewers on the North River, and a good many on the East River below Fifty-ninth Street, discharge their sewage well out in the stream. It has been found, as the sewage is carried out into the river here and there, so that it pollutes the water near the pier heads, that the teredo is not nearly so destructive, and it seems to be disappearing in the waters of the Hudson. It is still to be found there, but smaller and less ravenous, and fewer in number than a few years ago. Evidence is not wanting to show that this is so, and I may cite the case of the derrick *City of New York*. She was docked in the fall of 1878, and it was found that the yellow pine sheathing on her bottom and sides was almost riddled by the worms.

She has recently been on the dock, and I could find but very few holes in the sheathing after 15 years' exposure to the attack of the worms; and it was not removed, but left on her, being about as good

as new. I have taken occasion to examine a great many piles in the last few years, and I do not find that the worm is dangerously active at the present time, and it would seem as if the pollution of the water by the sewage was effective in stopping the growth of this pest.

Various protective experiments were tried by the Engineer-in-Chief of the Department of Docks, and I am permitted to give the following table showing their efficiency. In May, 1878, certain pieces of wood were put down at the end of Pier 1, N. R., in groups, as follows:

GROUP No. 1. SPRUCE, 4 x 10 INS., 12 INS. LONG.

No. 1. Plain, as cut from plank.
No. 2. Studded with 12 lbs. of carpet tacks.
No. 3. " " 8 lbs. of three-penny nails.
The surface of Nos. 2 and 3 entirely covered with nails.

GROUP No. 4. PART OF WHITE OAK PILE, 10 x 11 INS. DIAMETER, 31 INS. LONG.

No. 10. Plain.
No. 11. Studded with 35 lbs. of carpet tacks, driven about ¼ in. apart.

GROUP No. 5. PART OF PINE PILE, 11½ x 12 INS. DIAMETER, 36 INS. LONG.

No. 14. Plain.
No. 12. Studded with 20 lbs. of bellows nails and 12 lbs. of carpet tacks, driven about ¼ in. apart.

GROUP No. 6. PART OF SPRUCE PILE, 13½ x 14 INS. DIAMETER, 36 INS. LONG.

No. 15. Plain.
No. 13. Studded with 20 lbs. of three-penny nails, and covered on one side with zinc.

GROUP No. 7. PART OF SPRUCE PILE, 7 INS. DIAMETER, 36 INS. LONG.

No. 16. Studded with 4 lbs. of carpet tacks.
No. 17. " " 5 lbs. of three-penny nails.
In both cases driven from ½ to ¾ in. apart.

Group.	No. in group.	Description.	May, 1879.	May, 1880.	May, 1881.	May, 1882.	May, 1883.	Cost per square foot for protection.
No. 1.....	1	Spruce. No nails	Large number of worms.	Eaten more than in 1879.	Eaten more than in 1880.	Eaten more than in 1881.	Honey-combed.	
	2	Spruce, Carpet tacks.	No worms.	No worms.	No worms.	No worms.	No worms.	72 cents.
	3	Spruce. Three-penny nails.	"	"	"	"	"	38 "
No. 4.	10	Oak. No nails.	Large number of worms.	Large increase from 1879.	About same as in 1880.	Eaten more than in 1881.	Honey-combed.	
	11	Oak. Carpet tacks.	No worms.	No worms.	No worms.	No worms.	No worms.	80 cents.
No. 5.....	14	Pine. No nails.	30 to 40 worms.	50 to 60 worms.	Honey-combed.	Not as bad as in 1881.	About 180 worms	
	12	Pine. Bellows' nails and tacks.	No worms.	No worms.	No worms.	No worms.	No worms.	$1.00.
No. 6.....	15	Spruce. No nails.	30 to 40 worms.	60 to 70 worms.	Honey-combed.	Honey-combed.	Honey-combed.	
	13	Spruce. Three-penny nails.	No worms.	No worms.	No worms.	1 worm.	No worms.	43 cents.
No. 7.....	16	Spruce. Tacks.	1 worm.	1 worm.	1 worm.	1 worm.	No worms.	12 "
	17	Spruce. Three-penny nails.	3 or 4 worms.	9 or 10 worms.	About 17 worms.	About 17 worms.	10 worms.	10 "

RICHARD LAMB, Assoc. M. Am. Soc. C. E.—I think that the *teredo navalis* is somewhat of an epicure, and is not only looking for a home, but for something that it likes to eat. It seems to want wood that is acid. The charring of a pile gives a carbon coating that is not acid, and this it will not eat. If you leave the bark on the pile, the teredo will not attack it. There does not seem to be acid enough in bark to satisfy it. The place on the pile where the teredo will first attack will invariably be at the points where the charred wood or bark has been knocked off. The teredo chooses the woods that have the most acid in them. It will attack oak and pine before it will juniper or cypress, and it is generally known that it will scarcely ever attack the palmetto tree at all, which probably has less acid than any other wood.

I am somewhat interested in the Carolina Oil and Creosote Company. The experiments which the gentleman spoke of were made, and other experiments also in the same line. Ingredients of dead oil of coal tar were separated, and pieces of wood of similar size and texture were treated with each chemical extracted from the dead oil of coal tar. These pieces of treated wood were exposed at a place where the teredo is as ravenous as anywhere on the coast, and inside of one year every ingredient, except naphthaline, had been attacked. That seems to show that what we call "creosoting" is probably a misnomer, as the creosote itself was attacked, and the only chemical not attacked was the naphthaline.

Some of the gentlemen here who are interested in creosoting works know that when you buy dead oil of coal tar, if you specify the ingredients there will be a great difference in the price of one make of oil over another. I have noted a greater change for that containing an extra proportion of naphthaline. I think American engineers make a very great mistake in not specifying what their dead oil of coal tar shall contain. They call for "dead oil of coal tar," and there are just as many different kinds of dead oil of coal tar as there are gas houses. Most engineers seem to think that a piece of wood that will smell badly and look nasty is what they want. The best creosoted wood is of a dark leadish color, and the gummy substance on the surface of the wood which some engineers desire, and which I have reason to think some creosoting companies put on artificially, is not the effective treatment, as it after awhile leaves the pile by the action of the water. In the report of the American Society of Civil Engineers on the subject of wood preservatives, Mr. Andrews stated that he thought this gummy substance kept the worms from eating or boring into piles and was probably one of the main causes of keeping out the teredo. If this is the case my observation has shown me that the "main cause of keeping out the teredo" is removed in about one year. If creosoting is to stand on its merits it must be on the proper chemical properly injected into

the wood, and not upon a gummy substance which is soon washed off.

I made some experiments in filling the pores of wood with a material that would keep the teredo out, and that could not be washed off or dissolved by water. First I examined the teredo and found that the auger-shaped head with which it bores into the wood has a silicon point on it, so that it bores through the pile with glass. This suggested preventing its going into the wood by injecting glass into the pores of the wood. On reading up the subject of glass, I found that in England the walls of an old stone cathedral had been preserved from disintegrating, by injecting into the stone a solution of calcium chloride and silicate of soda (water glass). I succeeded in getting glass crystals into the pores of wood while at Norfolk, Va. The sample was exposed to the teredo, and in one year they had not touched it. Unfortunately I left the city, and the sample was subsequently lost. I have not since had time to experiment. I noted that after filling the pores with glass the wood did not increase in weight when soaked in water, showing that the wood did not absorb the water and would therefore be prevented from rotting. I also noted that the wood could not be burned. It would char, but not burn. It turned or dulled the edge of a blade when trying to cut it with a knife.

I think calcium chloride and water glass will probably be the chemicals with which the teredo will be kept out in the future. A little carbon mixed with it will make the glass very hard. He will be kept out mechanically by confronting him with glass harder than his glass auger, and not by poisoning him. By the use of calcium in this preparation the acid of the wood is also neutralized, making it less tempting as food for the teredo.

The teredo is a mysterious mollusk. If two planks are nailed together it will bore into one, and although it many come within $\frac{1}{16}$ in. of the other plank, it will not enter it, but will make its course often for many inches parallel with and exceedingly near the board it did not enter. Is there any gentleman here that can explain how the teredo grows in bulk? It attaches itself to a pile a small spawn no larger than a pin's head, and bores many inches into a pile, and at all times has its hole completely lined with a firm calcareous substance. The first year its diameter may not be over $\frac{1}{16}$ in., but the entire length of its self-made tunnel will be lined with a shell. The next year the entire length of the same tunnel may be twice the diameter and still lined with a correspondingly large shell. The consistency of the teredo is about that of an oyster. How can you explain its driving its auger into a hard oak pile, with no harder muscle than a snail's body? On still summer nights I have heard them grinding their way into the wood, and the noise of their grinding would surprise you if you should put your ear to the head of a pile in which they were at work.

JOHN A. MCDONALD, M. Am. Soc. C. E.—Mr. Thomson was saying that the teredo here attained $\frac{5}{8}$ in. in diameter; he does not give the length. In Australia we frequently find them $\frac{1}{2}$ in. in diameter, and the maximum length is about 3 ft.; a few years ago I took out two specimens of the teredo about 2 ft. long.

In regard to the teredo liking the timber, and in regard to its going into the timber to make a home there, there is generally some sense in the instincts of animals, and while the teredo will go through a small nail hole and will find its way through very small holes, yet it must know that it must grow and won't be able to come out again, and I take it is making the teredo out to be a bit more foolish than we ought to. We ought to think that the teredo goes in because it likes the timber, and it is willing to go in and spend the rest of its life feeding on the timber. The timber we use chiefly for piles weighs from 70 to 80 lbs. per cubic foot. As to the sample that Mr. Thomson passed around, I have seen in some of our northern rivers piles attacked as badly as that in 12 months' time. The accuracy with which the teredo will travel along a plank is wonderful; it will travel for yards close to a seam and within about $\frac{1}{16}$ in. of the seam and never go through it.

In regard to treating the piles, we never use creosote in Australian hard woods, because we cannot get the creosote into the woods; they are too dense; and we have to use external means. We generally use metal, and then replace any sticks which are attacked by the teredo. We have used the two modes described in the paper, that is, the iron tube and filling with concrete, and earthenware pipes.

H. COMER, Assoc. Am. Soc. C. E.—I merely wish to say that the two samples on the table were given to me some time ago; they are specimens of treated and untreated piles used in a railroad trestle and driven alongside each other for 15 or 16 years. The creosoting was done by Mr. Andrews, and I think they speak for themselves as to whether creosoting can prevent attacks from the teredo or not. I believe I am right when I say that when the American Society was making the investigations as regards the preservation of timber some years ago, it found that wherever treated or creosoted lumber had failed, and it could be traced out, it was proved to be through improper treatment, or a poor quality of oil had been used. The same result has been reached by our English friends. This, I am aware of, from pamphlets and different records I have read. I sent these specimens up to-night with the idea that they might be interesting to some of the members.

One of the members to-night said, in regard to specifications for dead oil, that that oil which had the most naphthaline in it was the most expensive. I can speak from personal experience, that it is the cheapest. The most expensive oil as used in creosoting on the other side is what is known as "country oil"; the country oil has a higher

percentage of tar acids and a lower percentage of naphthaline and is much more expensive. Mr. Bolton in his paper claimed that the combination of tar acids and naphthaline is what is so effective in preserving timber, and in his opinion the best oil was that which contained a high percentage of naphthaline.

H. W. BRINCKERHOFF, M. Am. Soc. C. E.—In regard to the question that has been raised as to whether the teredo goes into the pile for board or lodging, if the hole that it makes and occupies is very much larger than its own person, we might suppose that it has enlarged it for food and not merely for exercise. I would like to ask if any one has observed what proportion of the length of the tunnel made by the teredo is usually occupied by its body?

F. COLLINGWOOD, M. Am. Soc. C. E.—While connected with the construction of the dry dock at Newport News, located near the mouth of the James River, I was obliged to consider the subject of the protection of timber against the attacks of the *teredo navalis*, as the ravages of the worm are very extensive in those waters. A pile 1 ft. in diameter would be rendered useless for supporting heavy weights in the course of six months. Unfortunately the contract for the work had been signed before I was called upon, and the specifications required simply that the timber should be creosoted, but with no provision as to quality. The contractor had, therefore, arranged to have the creosoting done at Wilmington by the injection of wood creosote, which was prepared at the works by the distillation of yellow pine timber.

I made a personal visit to Wilmington and examined some small timbers which had been used as piles in Smith's Sound. These had been partially protected by the injection of the wood creosote oil, and had been in use a year. I found that while the protection was perfect, from high tide to some distance below, the worm had entered extensively near the mud line, and some timbers at another point had been eaten off. I brought back with me some of the oil, and requested an analysis, but it was never made.

At the coal docks of the Chesapeake and Ohio Railroad, at Newport News, some piles had been in use several years, which had been creosoted with dead oil creosote, after being seasoned. The creosoting was done by placing the piles in a tank containing the creosote, where they were soaked for about six weeks. These were not examined by a diver, but an examination at low tide showed no evidence of the entry of the teredo. It will be remembered that the English writer, Mr. Bolton, claims that the valuable principle in the dead oil creosote is especially the acridine; and on tasting the wood to which this creosote had been applied, it was exceedingly bitter, while the wood to which the wood creosote had been applied was sweetish in taste and almost devoid of bitterness. As the contract was allowed to be completed by

the use of wood creosote (which had received the endorsement of a board of officers appointed by the United States Government), I had an opportunity to see how it withstood the test. Upon examining the timber after it had been in place about a year, I found that the teredo had already begun to work on the angles of sticks near low water; and since that time extensive renewals have been necessary.

In reference to the question of the life habits of the teredo which has been under discussion this evening, I believe that it begins its life as a free swimmer, the same as the spat of the oyster, microscopic in size; it finally attaches itself to any wood surface and immediately begins boring and growing in length and diameter as it bores. The teredo has an influent and an effluent siphon, by means of which it takes in and discharges water, from which it abstracts the animalcules which form its food, and the carbonate of lime, with which it lines the hole it excavates. The original opening is never sealed, as the animal must have access to clean water, and dies as soon as it is surrounded by that which is stagnant or foul. There is no evidence that the wood excavated becomes a part of its food. The animal belongs to the class of borers, some of which bore into mud, some into shell, and some into stone, which certainly could not form food for the borer.

CHARLES MACDONALD, M. Am. Soc. C. E.—The paper and discussion furnish the best evidence that timber piles are not suitable material for permanent construction in "teredo" waters. It is a question whether it would not be even better policy to adopt iron construction at once, where from motives of economy, a low first cost is a necessity. This might be either in the composite form used at Old Point Comfort, and described by John B. Duncklee, M. Am. Soc. C. E., in the *Transactions* of this Society, Vol. XXVII, page 115, or in solid metal throughout.

In 1883 I built an iron pier at the Island of Oruba, South America, using solid iron piles, 5 ins. in diameter.

They were easily driven to a firm bearing in the coral formation of the sea bottom, and have thus far given satisfactory results.

L. J. LECONTE, M. Am. Soc. C. E. (by letter).—The results described in the paper agree very well with experience on the Pacific Coast harbors during the past 24 years. A great many attempts have been made towards creosoting piles as a protection against the teredo, but, generally speaking, the products put on the market have been very inferior, and consequently the piles soon became honey-combed. Of late, however, some good work has been done, and there are good reasons to believe that the future will develop a better record for creosoting work. Recent experience shows that in attempting to impregnate freshly cut or green timber with large percentages of dead oil, with the view of insuring perfect protection, another grave difficulty has presented itself. When the impregnation process is completed and the piles are hauled out from the boiler tanks, they are found to be

badly checked, some of the cracks ½ in. wide running nearly the length of the piles.

The sap wood also seems to be separated from the hard wood, so that when one kicks the pile it emits a hollow sound, similar to that of a rotten log. To prevent these cracks from running and opening during the process of driving, it becomes necessary to wrap the piles with telegraph wire, say six or eight round turns, at intervals of 6 ft. along the length of the pile. If they are to be driven in hard ground and this protection is omitted, the creosoted piles will splinter and break all to pieces under the blows of the pile-hammer. The life and strength of the timber seems to have departed. If any member can tell us how to creosote freshly cut green wood thoroughly, and at the same time not kill the life and strength of the fiber, we would be pleased to hear from him. The plans of using concrete and vitrified pipe for pile covering were both applied on works in San Francisco Bay some 20 years ago, with the same end in view, with this slight difference that the ~~creosote~~ was in the form of ordinary cement sewer pipe. The city front of San Francisco is exposed to very heavy storms from the southeast during the winter, and these coatings have failed to stand the shock due to driftwood driven by heavy seas. In sheltered places, doubtless, this is a good and lasting protection.

A still cheaper process is now being extensively used on the Pacific Coast for protecting piles between the mud line and high water-mark, namely, the Paraffine Paint Company's coating, consisting of a heavy coat of paint, then a jacket of heavy canvas saturated with the same paint and nailed on with copper-plated nails, and, finally, an outside protection of closely laid wooden battens, fastened on with wire nails. The latter is simply to protect the coating during the rough handling incident to pile-driving work in general. This process does not cost more than 35 to 40 cents per running foot of pile between the mud line and high tide, and has stood a severe test for eight years or more in waters where teredo and limnoria destroy ordinary piles and woodwork in from four to six months.

Another plan which has been extensively used on the Pacific Coast since 1867, and possibly before, consists in coating the timber with boiled coal tar; then a good covering of stout felting saturated with coal tar and fastened on with galvanized iron nails; and, finally, an outside protection of redwood battens or boards, as the case may require. The best test of this plan is that furnished by the facts connected with the construction of the United States Engineer's wharf at Lime Point, in the bay of San Francisco, near the Golden Gate. The foundations of this wharf consisted of Oregon fir cribs filled with stone. The timbers were square, the ends halved in and drift-bolted, so that the outside faces of the cribwork were tight jointed, smooth and continuous. These cribs were given an outside coating of coal tar and felting, as

above described, and finally sheathed with redwood boards. This work was done by Colonel George H. Mendell, Corps of Engineers, U. S. A., in the winter of 1867–68, and the woodwork is as sound now as the day it was put in, some 25 years ago.

The teredo and limnoria are very destructive in this locality, ordinary piles lasting five to six months only. This coating is now extensively used for wooden buoys, also to protect the bottoms of bay schooners, transfer lighters, dredging machines, etc.; in fact, is generally used for protection of all woodwork against the ravages of the teredo and limnoria instead of a covering of copper or composition metal.

The cost of this covering is very reasonable, say 5 cents per square foot all told. Some shipyards are now using heavy tarred roofing paper, instead of the more expensive felting above mentioned, and speak well of it; but sufficient time has not elapsed to place it on the same footing with good heavy felting.

Ernest Pontzen, Cor. M. Am. Soc. C. E. (by letter).—For many years I have been interested in the question of the preservation of timber. There are two methods of protecting timber against the *teredo navalis*, namely:

First.—Prevent the *teredo navalis* from approaching the timber.

Second.—Introduce into the timber poisons which will kill the teredo.

These methods can be used simultaneously. The introduction of concrete into the space between the piles and iron shells, and the use of vitrified clay pipe filled up with sand, are both methods under the first head. Impregnation with creosote or sulphate of copper comes under the second head, but the poisons, in order to produce the result required and to kill the *teredo navalis*, must be soluble; therefore, only soluble matters, which may be washed out by water, can be efficacious in the preservation of timber.

In order to prevent this washing out, it is well to combine the two methods of protection. The statement made in the paper under discussion, that "creosoted timber has been preserved from 1876 to 1886," proves that the useful parts of the creosote have been retained ten years without an outside coating to prevent washing out. In my opinion, it is quite sufficient to cover the creosoted wood with a thin plate of metal, or any other coating, in order to prevent washing out

of the soluble antiseptics. The use of a cement or concrete coating, as well as the use of vitrified clay pipes, appears to me to be insufficient when the timber has not been treated with poisons which will kill the teredo, because the finest cracks allow entrance to this very small animal. If the timber, however, has been treated with antiseptics, a less extensive outside protection to prevent the washing out can be provided.

If the timber has been treated with sulphate of copper, I think that the exposed parts could be subjected to electric action, which would produce galvanically a thin coating of copper. The timber can also be subjected after treatment with sulphate of copper to the successive action of a silicate or of sulphate of barium, producing on the surface a non-soluble combination. If the timber has been treated with creosote, I think it possible to cover it with layers of tar and sand which would prevent washing out.

As the parts of the timber exposed to the attack of the *teredo navalis* are those above the mud and between high and low water, there is no danger that this coating will be injured in the driving of the pile.

These ideas have not been sustained by practical tests, but it is my opinion that they could be, and I submit them to the judgment of the eminent engineers who are interested in the preservation of timber.

R. MONTFORT, M. Am. Soc. C. E.—The piles used on the Louisville and Nashville Railroad Company's lines were round, with the bark removed prior to treatment. The sap was allowed to remain on, as it absorbs a larger proportion of oil under pressure than is possible to force into the heart. When the bark is left on untreated piles it protects them from the teredo for a short time, but within two or three years they are destroyed. Charring would doubtless protect them in a similar manner.

The oil used in treating piles on the New Orleans and Mobile Division was manufactured in this country; the oil used in treating the wharves at Pensacola, Fla., was imported from England. The American oil has given better results. Samples of piles that had been attacked by the teredo were sent to Mr. S. P. Boulton, of London, Eng., in 1887, for inspection and analyses. His general conclusions were that the weight of the oil used (8¾ lbs. to the gallon) was not sufficient, and that oils of a heavier description and higher distilling points, rich in

green oil and the less volatile antiseptics, would have given better results.

After separating the oils from the wood resin, with which they had become incorporated, no less than 95% came over on distillation up to 600° Fahr. The lighter antiseptics are unreliable from their washing out or evaporating. He also considered the trouble, in a great measure, due to the limited time for seasoning the timber, which resulted in insufficient removal of the moisture before creosoting.

The fact that the piles resisted the attack of the teredo for from six to 14 years, and that 952, out of a total of 4 274, showed no signs of attack at the end of 14 years, would seem to prove conclusively that the oil used must originally have contained the proper constituent to protect against the teredo, and that the action of the waves gradually washed it out, thus allowing the teredo to attack the piles.

In recent years, we have been experimenting with a mixture of resin and dead oil, with the hope that the resin will aid in fixing the oil and preventing it from being dissolved or washed out. Sufficient time has not elapsed to form any definite conclusion, although so far piles thus treated have given satisfactory results. There can be little question but that it is practicable to construct piles of iron or steel that will be more durable than wood treated with creosote or other known protections. As to whether the metal piles are cheaper in the long run, taking into consideration the difference in first cost, with compound interest on the saving effected by using wood, is something that must be settled independently in each case and cannot be decided at once for all cases.

At Bay St. Louis and Biloxi trestles the piles average 50 ft. or over in length, and cost 30 cents per foot, $15 per pile; driving, per pile, $3; capping and bracing, $2; total cost per pile in place, $20. If 50-ft. iron piles 5 ins. diameter, such as those used successfully by Mr. Charles Macdonald in 1883 in the construction of an iron pier at Island Oruba, South America, were substituted for the creosoted yellow-pine piles, they would weigh 65½ lbs. per linear foot, or a total of 3 275 lbs., and would probably cost about 3 cents per pound in place, equal to $98.25. If we add screw end and cap, say, 1 000 lbs., at 3 cents in place, $30; for bracing, struts and ties, $21.75 in place, makes the total cost per pile $150. Difference in first cost in favor of creosoted wooden pile, $130; compound interest on this $130 in three years at 6% would

equal $24.83, or more than sufficient to entirely renew the creosoted pile. The creosoted piles at Bay St. Louis and Biloxi have been in use for 14 years, and with exception of the small percentage that was destroyed in the hurricane of October 2d, 1893, will probably continue in use for at least 10 years to come. The cost of the recent protection has been much less than it would have been to have replaced the existing piles by new creosoted piles. Even if the cost of the 5-in. diameter solid iron or steel piles could be reduced to one-fourth of that indicated by the foregoing figures, we would still be unable to show economy in their use as compared with creosoted piles. The creosoted piles average 14 ins. in diameter above the bottom of the bay. If we figure their strength as columns compared with the 5-in. iron piles, assuming both piles braced so as to fix them at points 12 ft. apart, measured along the piles, the pine piles are twice as strong as the 5-in. wrought-iron piles, while their transverse strength is nearly three times as great to resist blows from drift. It is true that the bearing power of a wrought-iron pile as against settlement can be increased indefinitely by increasing the size of the screw disc on which it rests, but no trouble from settlement has been experienced with the pine piles.

At Mobile River, on the Mobile and Montgomery division of the Louisville and Nashville Railroad, a bridge consisting of five fixed spans, each 140 ft. in length, and a draw span 259 ft. long, rests upon wrought-iron screw pile piers driven in 1871. The piles are made solid, 6 ins. diameter, and the construction of the piers is fully described in the *Engineering News and American Contract Journal* of April 4th, 1885, in an article by Col. Wm. M. Patton, C. E. The piles have done good service and are still in use. It has, however, long since become necessary to drive wooden piles in front of the piers and construct fenders to protect them against blows from the large drift that floats in the river. Recently, when rebuilding the superstructure of the bridge, the piers were strengthened. As to what the life of 5-in. solid wrought-iron or steel piles at Bay St. Louis and Biloxi would be is difficult to say, but creosoted pile piers at West Pascagoula and Pearl River, where the water is not nearly so salt, were originally braced with 1¼-in. round rods; many of these rusted completely off within ten years. Also a ⅞-in. wrought-iron drum on the center pier of Biloxi draw rusted entirely through.

The *Engineering News* of February 8th, 1894, contains a very interesting article on teredo-proof sheathing for piles used on the Pacific Coast, and a report of Mr. Howard C. Holmes, Chief Engineer of the California Board of Harbor Commissioners. Among others, he describes the method used by A. W. von Schmidt, which consisted of a loose jacket of earthen sewer pipe, with the joints and interior space filled with hydraulic cement. This did not give good results, on account of blows from drift and cracking, due to want of elasticity. The pipe at Bay St. Louis and Biloxi is extra strong, and the joints are made with a composition of coal-tar pitch and sand, which is more elastic than cement. The space between pipe and pile is filled with sand. Drift is practically unknown, except in rare cases of extraordinary hurricanes. Mr. Holmes also describes methods of coating the piles with asphalt, marine cement and Portland cement, applied with a brush, three coats before driving. None of these methods at all correspond to those used by the Louisville and Nashville Railroad Company.

The Paraffine Paint Company's process and the coal-tar process used at Lime Point, described by Mr. L. J. Le Conte, are not applicable to piles already driven and which have been in use for a number of years, such as those in Bay St. Louis and Biloxi trestles.